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A NEW ENGINEERING MUSEUM IN BERLIN, GERMANY.

By BRUNO BRAUNSBURGER.*

It is several years since the railway between Berlin and Hamburg was purchased by the Prussian State. As its Berlin terminal was not sufficient to handle the growing traffic a fine new station was built in the northwestern part of Berlin and the old one remained unused for a few years. Then the authorities decided to utilize the old building as a museum for railway and marine engineering, and, after several years preparation, it has been opened recently. Thus the country which is already rich in educational institutions has a new one which, owing to its character, is the only one in the world. As the name indicates, "*Kgl. Verkehrs-und Baumuseum*," it is confined to railways, water construction and transportation, and architecture. It is only a few months that all departments have been opened to the public.

The exterior of the former station (Fig. 1) has been left unchanged, but extensive alterations were made inside, especially in the cellar, where a power plant for lighting, heating and compressed air has been erected. The latter is used to operate certain models for demonstration purposes. In an adjoining yard are certain objects which, on account of their natural sizes, were too large for the inner halls.

The museum contains three classes, of which the first, concerning railways, is the largest. It can be subdivided into eight groups. The catalog gives no less than 6,000 objects altogether, four-fifths of which are costly models, the rest being photographs, drawings and literature.

Near the entrance there is a collection of rails, sleepers, switches, and crossings from the earliest to the latest times. Samples of material for track maintenance and tools are also shown. For the expert, sections of worn-out rails and sleepers will prove instructive, also a special testing plant demonstrating the influence of shocks on roadbed and support due to engines and heavy cars. Drawings show the increase in weight, length and height of equipment since 1842. Models of safety barriers, fire-protection strips, and of drains where a country road crosses the railway are placed on tables. They would be an object lesson to the American engineer, in whose country safety gates with watchmen, outside the cities, are unknown.

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The English system is admirable, as grade crossings are avoided wherever possible, while in Germany barriers are placed, with gatekeepers, everywhere in the whole empire. This is, of course, a heavy expense, but the roads here do not serve selfish interests, but the comfort and safety of the patrons. These features count more than profit, which is also the case in all other European countries.

In the second department, civil engineering work in connection with the construction of railways is exhibited. There are whole stretches of tracks, stations, administration buildings, car barns, water towers, coaling plants and the like. A prominent place has been given to a model of the Altona station, a city near Hamburg. It represents a typical German terminal and is of particular interest because it contains a power plant for an electric local line. It also illustrates the principle of avoiding grade crossings as far as is possible. Many photographs, maps and diagrams contribute to the completeness of this group.

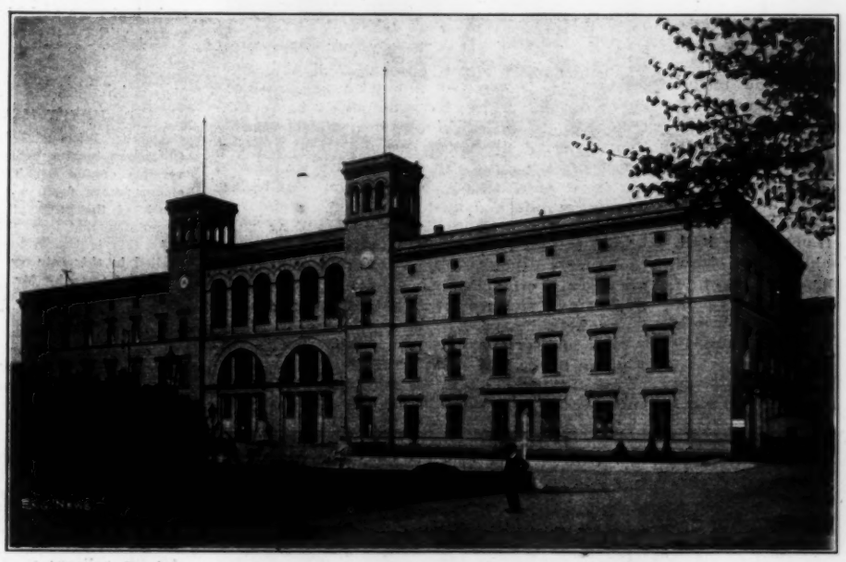

FIG. 1. VIEW OF ENGINEERING MUSEUM, BERLIN, GERMANY.

The next group is devoted to a field in which the lead of Germany is recognized. This fact induced the Government to erect an exhibit at the St. Louis Exposition, in 1904, of signaling and safety devices. A large space has been reserved for the 500 objects belonging to this department, such as switches, signal arms and shunting appliances, operated by hand, compressed air and electricity. Telegraphic and telephonic apparatus are also seen. One gets an insight into the wonderfully developed block-system, which foreign roads are adopting more and more. Models for both single-track and double-track lines and electrical alarms, track telephones, etc., are exhibited in a condition ready for operation, while the advance made is shown by early types. A fully equipped shunting plant and house has found its place in the adjoining yard in the open air.

The largest and most interesting department is the fourth, comprising locomotives and cars, which has received the most prominent location. There are a number of old and new engines in glass boxes. The oldest is the "Adler," of the first German railway, Nürnberg-Fürth, 1835; another is the "Borussia" of the Cologne district, from 1858 (see Fig. 2). Besides general modern engines there are some for special uses, mountain railways, industrial purposes and snow plows; also separate tenders, trucks, boilers and accessories, valves, whistles, measuring apparatus, fire-boxes, rivet-joints and tubes. Conspicuous, on account of its full size, is the cab of a modern express locomotive, to the platform of which steps lead to allow the public close inspection of its many intricate parts. The same applies to the smoke box of a ¾ coupled engine, with a "Schmidt" superheater. Not far from this is a worn-out boiler from the year 1881, cut open longitudinally to show the interior location of the many pipes, and also the destroying action of fire and water. The cars are still more fully represented; we see those for the first, second, third and fourth class, for mail, baggage, funerals, emergencies, hospital, dining, sleeping and parlor cars. Fig. 3 represents a third-class passenger car, of 1843, with wooden benches and buffers. How goods trains are made up is shown also by many models, the appearance of the vehicles depending on the character of freight. Thus we notice cars for long tree-trunks, rails, boilers, guns, chemical products, beer, fish, cattle and the like. Single parts are also shown, breaks, axles, wheels, couplers, trucks, lighting apparatus for oil, gas and electricity, a steam turbine with dynamo, heating and ventilating appliances. Very interesting are two brake batteries, each of which consists of 20 brakes of the Westinghouse and Carpenter design, respectively. The operation of the brakes in a train of 20 carriages is thus clearly illustrated. The power for such demonstrations is taken from the compressed air plant in the cellar, already mentioned.

As in our modern times, Germany, like some other industrial countries, is seriously experimenting with electricity as a motive power, even for long-distance trains, a special room has been reserved for this class. Owing to many presents

made by the two leading electrical firms of the empire one can follow the development from the early beginnings. No less than 500 objects are exhibited, comprising overhead and underground conduits, insulators, feeding cables, connections, details of rolling stock, safety devices, etc. Very attractive is the model of that famous high speed car which, during the trials on the Berlin-Zossen line, obtained the previously unheard of speed of 125¾ miles an hour. A photograph of the original car is here reproduced, as Fig. 4.

In the next group workshops and machine tools are exhibited. The government does not build any cars or engines, but it runs several large shops for the repairs of the rolling stock; such a model shop of the latest type is that in Opladen, of which there is a relief model (Fig. 5) in the museum. Of machines there are lathes for locomotive wheels, and crankshafts, steam hammer, hydraulic wheel-press, jackscrew, portable crane, traveling platform, tilting apparatus for coal vans and the like. In a larger proportion is the ventilating plant of the Kaiser Wilhelm Tunnel, near the Rhine, which can be shown in action. The same applies to most of the machines, which have been just named. Some ferry boats of old and new style are exhibited.

How railways are managed, and passengers and goods are handled, can be seen in the next group. There is a store of printed matter, diagrams, etc., with rules for the station master, gate-keeper, switchman, guards; also their tools and uniforms. Worth mentioning is a fully-equipped ticket office, with a stamping machine, automats, baggage scale and elevator. An historically valuable collection of tickets, dating back to the earliest times, is kept in a locked album.

A matter in which Germany again leads is its care for employees. Several relief models of invalid homes, dwellings for workmen and officials, hospitals, medical apparatus, beds, uniforms, food products, disinfecting appliances and the like are exhibited, with printed matter and photographs. Diagrams show railway income and expenses during the last 50 years, illustrating the huge increase in net revenue. Remarkable is an obelisk, consisting of three large blocks, each of which would represent, in pure gold, the mass of money spent for sick, injured and invalid employees, making a total of 131,451,321 marks ($32,000,000) during the decade from 1895 to 1904. This department would no doubt be an object lesson to American railway companies, most of whom spend nothing whatever for philanthropic purposes. This is the more surprising as American railways earn twice as much as the railways of Europe.

In concluding the review of this in-every-way exhaustive section, brief reference may be made to the water department, which is housed in the eastern wing of the museum. It comprises construction work for rivers, lakes, artificial waterways, harbor and seacoast. There are models of sluicegates, weirs, dams, viaducts, bridges, sections through streets; also vessels, dredges, icebreakers, fireships, etc. Two electric haulage locomotives are shown and also models of light towers, big reflectors, fog horns. Worth mentioning is a relief model of the island Helgoland.

In the adjoining three rooms civil engineering and architectural work is exhibited. There are some fine examples of German state buildings, exteriors and interiors, models as well as photographs. Transparencies and paintings are hung up in various rooms of the building. These contribute to the artistic appearance of the institution. Internal appliances for heating, lighting, and ventilating are shown, and also samples of materials, as sandstone, marble, stone—and glass mosaic, granite and linoleum, etc.

Of recent date is a library and reading room where, besides the current journals and technical books, very valuable documents are stored, which are mostly originals. Also much other printed matter and pictures relating to the exhibits are found here.

Thus the German public can claim another fine addition to the long list of magnificent educational institutions. This is the more pleasant as the engineering profession did not find until very recently the appreciation it deserved. The public gets an insight into such important technical matters as land and water transportation and certainly must be impressed by the ingenuity of the men who have created such wonderful things. The British or American engineer, although his education is considerably lower, is much more respected in his country than his German comrade. Happily this unbearable state of affairs is coming to an end and a right appreciation of the technical man and his work, as is seen in this museum, will be the result.

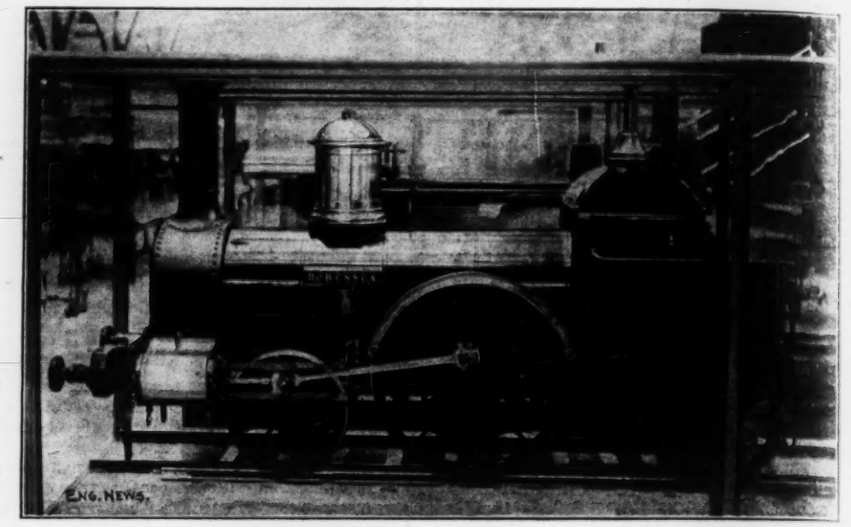

FIG. 2. A GERMAN LOCOMOTIVE OF 1848.

FIG. 3. A GERMAN THIRD-CLASS PASSENGER CAR OF 1843.

AN EXHAUST STEAM TURBINE PLANT.*

By HENRY H. WAIT,† M. Am. Inst. E. E.

At the Wisconsin Steel Company's Mill at South Chicago a turbine utilizes the exhaust steam from a reversible engine which drives the blooming rolls. The steam passes first to the receiver which takes out the shock of the puffs of steam, thence to the steam accumulator, and from there to the turbine and condenser.

As this is the first plant of this character to be installed in this country, it was subjected to an elaborate series of tests by Mr. A. U. Leonhauser, the chief engineer of the Wisconsin Steel Co., and Mr. F. G. Gasche, mechanical engineer of the Illinois Steel Co. Besides testing the turbine equipment, the arrangement offered an opportunity to test the steam consumption of the primary engine by making a temporary change in the piping, so that the exhaust was led directly to the condenser without passing through the turbine.

PRIMARY ENGINE.—This is a 42 × 60-in. double-cylinder engine, of the rolling-mill type. When in normal operation the engine rolls about 10 ingots per hr., with 21 passes per ingot, stopping and reversing after each pass with a short interval for the starting of a new ingot, so that it is stopped or practically idle 20% of the time during the cycle required for turning out an ingot. There are other frequent stops for the ordinary manipulation of the mill, which last from a few seconds to several minutes.

The average I. HP. while the engine is actually running, is 1,010, and figuring back from the total steam consumption, gives 54 lbs. steam per average I. HP. during the running period. This large consumption is readily understood when we consider that the engine takes steam for practically the full stroke when starting the passes, and is running on very light load most of the rest of the time. Of course the horse power, and especially the torque, runs up enormously when the rolls first take hold of the ingot.

RECEIVER.—This auxiliary is to relieve the accumulator of the stresses and disturbances which would occur if the steam went directly to it. The receiver consists of a tank with a number of baffle-plates and is provided with drains for water and oil. At the top of the receiver, between it and the vertical exhaust-pipe, used when desired to exhaust directly to atmosphere, will be seen the relief valve which permits the escape

*Abstracts from a paper before the American Institute of Electrical Engineers, New York City, Dec. 13 1907. In our issue of Sept. 29, 1904, several types of heat accumulators, for exhaust-steam turbine installations, are described.

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of steam to the air during the periods when there is more than the demands on the turbine and regenerator require.

PRINCIPLE OF ACCUMULATOR OPERATION.—The accumulator might be called a heat fly-wheel, absorbing or giving up energy in accordance with the requirements. The accumulator is virtually the same as the more recent Rateau regenerators built in Europe.

When the engine is running, the exhaust steam comes from the engine through the receiver and is delivered to a number of pipes immersed in the water in the regenerator. These pipes or ducts are perforated with a number of small holes, spraying the steam, through the mass of water in the regenerator. A greater or less proportion of this steam is condensed in passing through the water and gives up heat to the mass of water in the regenerator. It is usual to operate such accumulators at about atmospheric pressure. In other words, the steam coming to the regenerator will usually have a temperature of 212° F. and will tend to heat the water to just that temperature. If the engine stops and the supply of exhaust steam discontinues, we will see that we have a large mass of water heated to 212° F., and if there is a continuous load on the turbine the flow of steam through the turbine to the condenser will tend to make the pressure fall off slightly in the accumulator, and 212° F. will then be slightly above the boiling temperature of water at this lower pressure, so that the mass of water begins to give off steam and act like a boiler running at approximately atmospheric pressure. If, now, the engine starts again, steam will be delivered to the accumulator at a temperature slightly above that to which the water has fallen, due to the cooling effect of the evaporation of the steam for supplying the turbine, and the mass of water will again absorb heat from the exhaust steam.

In actual practice it is more convenient to run the accumulator at a pound or two pressure above the atmosphere, as in this case the piping is not under vacuum so that so much care does not have to be exercised to avoid air leaks. However, in certain cases, it is desirable to run below atmospheric pressure. In this way the power of the primary engine may be augmented by letting it operate at a partial vacuum. Plants are actually running with a delivery pressure to the turbines as low as six pounds below atmosphere.

DETAILS OF ACCUMULATOR.—The accumulator at South Chicago, being quite a large one, is divided, by a diaphragm in the middle, into two decks, each deck having a series of flues. The steam generated in the lower deck passes up through steam flues into the upper chamber and passes out with the upper steam through the steam dome. There is a small percentage of the steam delivered which is condensed on account of radiation from the surface of the accumulator. This makes an accumulation of excess water which is automatically discharged by a float trap, at the end of the accumulator. In most plants where such apparatus is used, there is more exhaust steam than is actually required for the turbines, so that this condensation does not matter, as it is only a small percentage anyhow. But in some plants, where the fullest possible amount of steam needs to be saved, the regenerators are covered with some heat insulator. Gage glasses are on each deck, and a series of valves and pipes are arranged so that the water level in the upper deck can be set to suit that maintained in the lower deck by the float valve. If for any reason the primary engine shuts down for a considerable period, the supply of heat stored in the regenerator will become exhausted and the pressure will fall below a workable amount for the turbines. To take care of this condition, there is an automatic reducing valve which is set so that it will open whenever the pressure falls below atmosphere and deliver live steam through the reducing valve to the regenerator whenever the primary engine is shut down for a long enough period to make it necessary.

In this plant the relief valve is set for three pounds above atmosphere; whenever there is more steam than is necessary to run the turbine and to heat the water in the regenerator to a temperature corresponding to its pressure, the excess steam will pass off to the air through the relief valve. On the other hand, if the engine does not give enough steam to supply the needs of the turbine, the reducing valve will open and let in enough live steam to make up the deficiency.

OPERATION.—At the Wisconsin Steel Company's plant, during normal operation, the pressure ranges about one or two pounds above atmosphere; when the engine is exhausting heavily it runs up to about three pounds. The lower limit of pressure, when the reducing valve opens, is about atmosphere.

As a matter of fact the accumulator is considerably larger than would be really necessary to regularize the flow of steam to the turbine, as tests show that under the ordinary load conditions the accumulator will keep the turbine running at its average load for about seven minutes after the primary engine shuts down before the reducing valve opens. This would correspond to a period of about five minutes at full load on the turbine.

In comparison we can take the low-pressure turbine plant of the Wisconsin Steel Company as an example. The primary engine delivers to the accumulator an average of 52,400 lbs. of steam per hr., and develops 1,010 I. HP. The tests on the turbine showed that it took 33.6 lbs. per electrical HP.-hour. This would make available at the switchboard 1,500 HP., or a total of 2,370 HP., after deducting the engine friction; that is, 22.1 lbs. of steam per total HP. at the shaft.

In general, it will be found to work out that the combination of a low-pressure turbine with a high-pressure rolling mill engine is a more economical unit than a condensing engine alone; and it frequently figures out to a lower steam consumption than a high pressure and low-pressure turbine or a high-pressure condensing turbine.

PRACTICAL RESULTS.—The general result of the installation of this low-pressure turbine equipment is that it enabled the mill to shut down the two 250 kilowatt engine-driven generators which formerly operated the mill, and for a long time the turbine carried the entire electrical load of the steel mill, operating from the exhaust of the blooming engine and not taking any live steam except during abnormal stoppages of the blooming engine.

Recently the company has installed some electrical unloading machinery on their docks. When this machinery is all in operation the load frequently runs up above the ultimate capacity of the turbine and it is necessary to start up one or more of the engines and run in parallel with the turbine.

The attendance and lubrication items for the turbine plant are very small. The turbine is located near the blooming engine, while the other generators are located in the blowing engine house about a quarter of a mile away. There is but one attendant on duty in the turbine engine room.

At this plant the boilers are supplied principally by gas from the blast furnace, but the supply of gas is quite variable and usually not adequate to give all the steam required, so that more or less coal has to be used. The installation of the turbine, therefore, results in a saving of the coal corresponding to the steam required for operating the dynamo engines. This, as indicated above, is quite a variable quantity, but has been variously estimated at from $10,000 to $20,000 per year. In figuring on the installation of the turbine plant, it was estimated that the turbine would effect a considerable saving, even if the supply of gas were generally adequate, as the maintenance of the turbine plant would be considerably less than the corresponding engine and boiler plant, or even a gas-engine plant.

Further, it is well to remember that exhaust steam passing to the atmosphere can be looked upon as the equivalent of a water-power plant and that it usually has the advantage of being near a market for power, besides costing less to develop than a water-power plant would. In other words, the exhaust-steam plant should be able to produce the power cheaper than a corresponding water-power plant.

During three months when the steel plant was run-

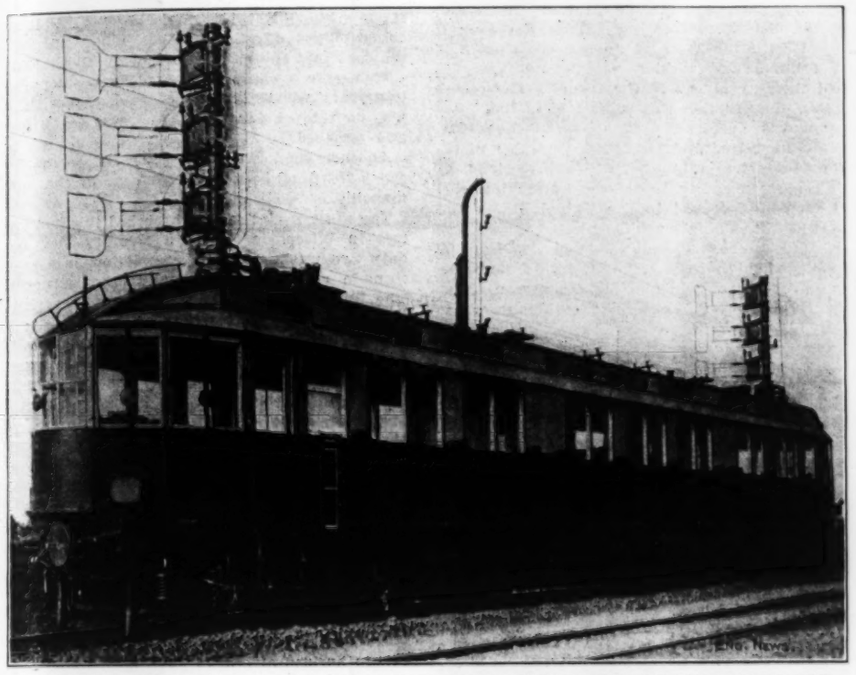


FIG. 4. THE HIGH-SPEED CAR USED IN THE BERLIN-ZOSSEN RUN OF 125¾ MILES PER HOUR.

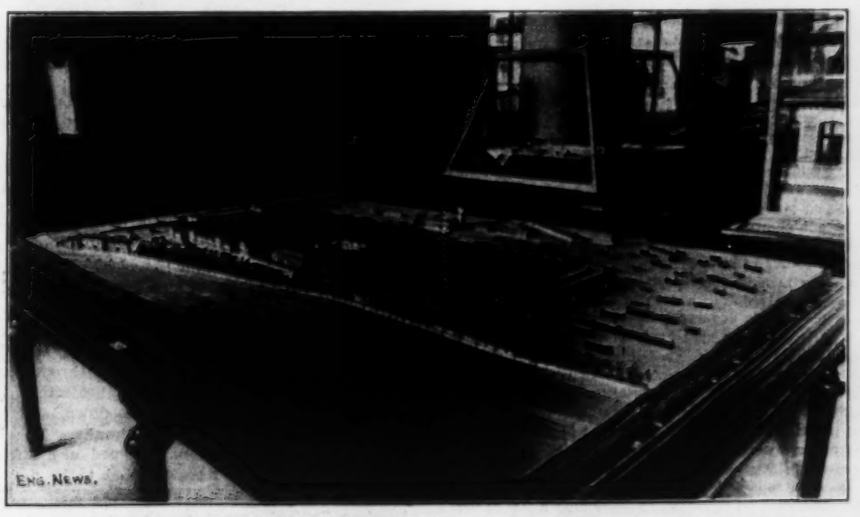


FIG. 5. MODEL OF PRUSSIAN GOVERNMENT RAILWAY REPAIR SHOPS AT APLADEN.

ning at nearly full capacity, the turbine delivered an average of 188,300 KW.-hr. per month, or 51 per cent. of the total possible kilowatt-hours if run at its rated load the entire time.

The operating expenses are at the following rates, based on the above output:

Oil, waste, etc.	0.002	cents	per	kw-hr.
Attendance	0.074	"	"	"
Maintenance and miscellaneous	0.011	"	"	"
Total operating	0.087	"	"	"
Fixed charges	0.212	"	"	"
Total cost	0.299	"	"	"

The fixed charges are figured on the basis of a cost of $80 per kilowatt. This figure would, of course, vary considerably with the conditions, but it can be taken as an average for moderate-size plants. Interest, depreciation, etc., are allowed for at 12 per cent. Nothing is allowed for superintendence, as no additional force is required for this item.

The cost being made up so largely of fixed charges, it varies very markedly with the load-factor. In fact, if the plant is run 24 hours a day, the lubrication, attendance and maintenance are only affected to a slight extent by the amount of load, so that they have almost the same effect as a fixed charge. Of course if the plant were run only during the day shift, the operating expenses would go down.

It will be noted that the cost of power at the larger load-factors is extremely low and would be even more so in a larger plant where the first costs and the other costs would go down considerably.

If the exhaust steam turbine plant were used for electric smelting or similar purposes, it is probable that the load-factor could be kept up over 80 per cent.

It might be remarked that the electric smelting processes take, on a very crude average, one kilowatt-hour per pound of metal produced. It will therefore be seen that such plants offer a good opportunity for doing certain kinds of metallurgical or similar work.

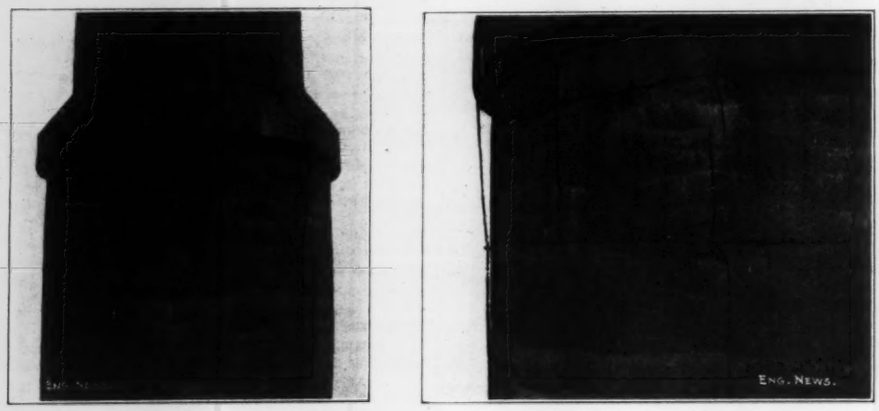
In the foregoing no value has been assigned to the exhaust steam. In most places where such exhaust steam occurs, there is not a good opportunity to utilize any considerable portion of it in any other ways, so that it would otherwise be a waste product. Under these conditions the question of the continuousness of the power would arise. Of course when the primary engine shuts down for more than a few minutes, there is a tendency for the boilers to blow off at the safety valve, so that for short periods it does not mean any more fuel, even if the primary engine is stopped. If the primary engine is stopped altogether, the boilers stand ready to furnish the steam to the turbine and the latter would consume approximately the same amount of steam as non-condensing engines would. If this condition of running with live steam were one that would occur for a considerable time, it would be advisable to install a mixed high-pressure and low-pressure turbine; that is, one which would have a high-pressure section which is automatically fed with high-pressure steam whenever this becomes the normal source of steam. Such plants as this are already operating in a number of places where the primary engine runs only during a day shift and it is necessary to have the electric power day and night.

The foregoing remarks apply more particularly to the conditions in a steel plant, but similar conditions occur with large mining hoists; of course any other source of exhaust steam, whether intermittent or not, can be utilized in similar fashion. It has been already pointed out that a great saving can be effected in connection with continuous running engines, and that such a system can be used to increase the power of such engines, and that, even when run condensing, the power and total economy can sometimes be increased by combining a low-pressure turbine and engine.

FAILURES OF REINFORCED-CONCRETE CHIMNEYS AND RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION.*

By SANFORD E. THOMPSON,† M. Am. Soc. C. E.

[The subject of reinforced-concrete chimneys has repeatedly engaged attention in the past few years by the complete or partial failure of such chimneys. Several chimneys have been blown over, others developed weaknesses even before they were finished and in consequence had to be taken down and rebuilt, and still others have cracked more or less seriously. The association of American Portland Cement Manufacturers, therefore, delegated Mr. Sanford E. Thompson, M. Am. Soc. C. E., to investigate the general subject with the view of finding the causes of the failures and making recommendations toward their avoidance in the future. The report on this investigation was submitted to the Association very recently. It appears that Mr. Thompson visited and examined a number of reinforced-concrete chimneys, and has received direct report on nearly 150 of the (approximately) 400 such stacks built in the United States and Canada. These stacks are distributed through all parts of the country, and range in size from 50 to 352½ ft. high by 4 to 18 ft. inside diameter. None are very old; in fact Mr. Thompson states that the first reinforced-concrete chimney was built no earlier than 1898, being one for the Pacific Coast Borax Co., at Bayonne, N. J., designed and constructed by Ransome & Smith Co. There has been sufficient experience with such chimneys, however, to lead the author to the general conclusion that the recorded failures are chargeable to poor workmanship, faulty design, or wrong concrete mixture, or all three; and that, while reinforced-concrete is a suitable material for chimney construction if the principles and methods proper to other types of reinforced-concrete work are followed, defective methods are employed in many instances at present. More specific recommendations to proper precautions in design and construction are given at the close of the following abstract; most of them merely state the essentials of good practice in all kinds of reinforced-concrete work.—Ed.]



TWO VIEWS OF A CRACKED CHIMNEY.

FAILURES OF CHIMNEYS.

Eight cases of failure have been reported in which chimneys have either blown over or have been taken down because they were defective, and ten or twelve others have been heard from in which cracks have developed which cause serious apprehension. In nearly every case of failure the chimney has been rebuilt by the construction company.

One of the failures most widely known is that which occurred at Peoria, Ill., in 1906. The chimney had been completed about three weeks when it gave way at the offset or projection, the upper part crumbling as it came down, and the concrete breaking into small chunks. The T-shaped steel reinforcement stripped clean from the concrete. The cause is stated to be a poor batch of concrete at the offset, although the manner of failure would indicate that the concrete in the upper portion also was not of the best quality.

Another case is cited in which a chimney stood for nearly two years and then blew over in a wind storm of about 45 mi. per hr. Here as in the other case the concrete stripped from the steel, and the fall was attributed to poor adhesion between the concrete and steel.

A stack built in very cold weather in Canada blew over when the concrete, which evidently froze without setting, thawed out.

A case occurred in 1905 where the upper 30 ft. of a chimney broke and slid off while it was being topped off, the accident being charged to the fact that the mortar had not properly set.

This year a chimney in the West, after about two years' service, developed such cracks that it was considered dangerous, and was taken down and replaced.

Another stack was torn down before the boilers were fired because of defects in the workmanship.

The most recent failure occurred this last summer in Wisconsin, where a chimney blew over, only two weeks after its completion, in a severe tornado which damaged many other structures. The blame is laid upon the unusual severity of the storm and the freshness of the concrete.

CRACKING.—Considering the chimneys which are now in commission, we find that in general they are subject to more or less checking or cracking. This need not necessarily condemn the structure, since the reinforcement may be sufficient to safely hold together the blocks formed by the cracks, and yet it would seem that, with the opportunity we have of introducing steel wherever needed, all cracking ought to be averted, especially as there is always danger that the cracks may increase from wind vibration, heat and frost.

The reports indicate in most cases that the cracks are not considered dangerous by the owners of the stacks. In several instances, however, long vertical cracks have appeared, and in others horizontal cracks have been found in the lower portion which have given serious concern. One chimney, straight when built, has since leaned about 3 to 4 ins. from the vertical, beginning at a point about two-thirds way from the base. Another informant reports soft spots in his chimney.

Photographs of a portion of a chimney which has developed cracks of a somewhat serious character are shown in half-tone reproductions herewith. The pictures clearly show the 3-ft. section constituting a day's work, and also, by the wavy lines, approximately horizontal, the joints between the layers in which the concrete was placed. The irregular line just below the offset represents a horizontal crack extending all around the chimney, and there is a continuous crack at the second section joint below this, as well as a number of vertical cracks through the offset extending in at least one case clear to the top of the chimney.

About 400 reinforced-concrete chimneys have been built in this country, and direct reports have been received from nearly half of them. The failures cited amount to about 2% of the total number, with at least 2% to 3% more of doubtful safety; it is probable that most of the seriously defective cases have been brought to light, since special care has been expended in running down doubtful ones. Of the other chimneys, even where personal examination has revealed somewhat serious cracks, the general verdict of the users is "satisfactory" and "good."

The vital question is whether the unsatisfactory cases may be passed over as isolated cases of defective construction, or whether they afford an arraignment of other chimneys now standing; and whether they predicate a similar percentage of defective construction in the future.

A CASE OF BAD WORK.—An examination by the writer, of a chimney which was being taken down, if representative of other chimneys throws considerable doubt upon their durability. On the other hand, it affords means for pointing out definitely the errors which must be guarded against

*Abstract of a report prepared for the Association of American Portland Cement Manufacturers.

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in the future. This chimney showed soft spots in three of the sections, where the concrete could be readily loosened clear in to the steel. The concrete was porous throughout, and scarcely bonded between the 6-in. layers. Samples cut from good portions of the concrete, which was a mortar of 1 part cement to 3 parts sand, gave an ultimate strength of about 1,200 lbs. per sq. in., or about one-half the strength of a good 1:3 mortar laid with a sufficient quantity of water. A month after this inspection the chimney was taken down, the 6-in. layers being readily loosened from each other and cut in pieces by a pneumatic chisel, and the concrete was found to vary materially in hardness. Of special importance was the fact that the mortar in the angles of the T-bars was weak and porous, showing scarcely any adhesion to the steel. Inquiry showed that the concrete or mortar used in the construction was of such *extremely dry consistency* that even where well rammed the moisture did not cover all parts of the surface of a layer nor did it produce a proper bond with the steel.

From such evidence it is possible to avoid the conclusion that other chimneys built by similar methods, with so dry a mixture as to give a weak, porous concrete and to provide insufficient adhesion to the steel, are likely to cause trouble in the future.

The defects quoted are not inherent in chimney construction. In other words, from a practical standpoint, it is necessary and quite possible to follow in chimney construction the methods which have been proved necessary for success in other structures made from reinforced concrete. As a proof of this we may simply point to the indisputable fact that concrete chimneys have been satisfactorily built with a proper factor of safety and with a wet mix which insures a positive bond to the steel, and with a dense concrete which protects the steel from corrosion and permits the bonding of the various parts of the structure. Since reinforced concrete is everywhere being constructed on these principles, and is proving durable and satisfactory, and is being used under all conceivable conditions, we have ample cause for security in reinforced-concrete chimneys provided they are properly designed and constructed.

EFFECT OF HEAT UPON THE CHIMNEY.

In the preceding discussion the effect upon the chimney of the interior heat from the boilers has not been directly referred to. Just what part this has played in the faults which we have noted is uncertain. Undoubtedly the interior heat adds to the stress in the concrete, and thereby increases the tendency to crack, especially at points near the top of the inner lining. This simply indicates, however, that the quantity and arrangement of the steel reinforcement should be adapted to resist this extra stress.

The interior heat affects the shell because concrete is a poor conductor. The interior surface for a depth of an inch or two is heated very much better than the exterior surface, and so tends to expand and crack the colder outside surface. This effect is most marked upon a thick wall, the action being similar to that of a thick glass bottle, which breaks more readily when hot water is poured into it than does one of thin glass. The stress or pull on the outside surface must be met by increasing the amount of circular steel and placing it near to this outside surface.

The effect of heat upon the concrete material itself is also a point which must be considered in chimney design. In the earliest chimneys built the concrete lining extended the full height while in later ones it has been generally carried up to only about one-third the height. Usually the lining has been reinforced concrete, although in some cases firebrick has been used. A few chimneys have been built with no lining at all.

While much remains to be learned with reference to the effect of heat upon concrete, it is known to be a most excellent fire-resisting material, although it has been found that a temperature as high as 1,500° F. continued for only two or three hours will draw out the water of crystallization so as to take out the strength for a depth of ½-in. to 1-in. Lower temperatures affect the material less, and tests at the Watertown Arsenal indicate that a good cement mortar will not be appreciably injured at 600° to 700° F.

Tests of actual chimney temperature are extremely meager, but from records available we may say that the temperature in an ordinary chimney seldom exceeds 700° at the base, while 400° to 500° is more usual. It is a fact not universally known among engineers that the temperature in a chimney remains quite high even in its upper portion. For example, in the test of one chimney the temperature at three-quarters of the height above the base ranged only 10% to 20% lower than at the flue. This makes it evident that if the lining extends only one-third of the way from the bottom, the design of the concrete shell above it should be adapted to resist considerable heat, while greater safety may be insured by extending the lining far above the lower third.

No reports of injured lining have been received. Many users have never examined the interiors of their chimneys, but several have reported that the lining was in good condition. One correspondent states that after three months' use the interior surface of the chimney is smooth, "without cracks," and that "the soot does not adhere to the surface but falls to the bottom or is carried out by the draft." Since the fire-resisting quality of concrete increases very greatly with age, it is fair to assume that if the interior surface is sound at the end of the first two or three months it will not disintegrate after that time. Right in this connection, Mr. E. L. Ransome reports a recent examination of the inner shell of a chimney built nearly ten years ago of a true concrete of cement, sand and broken stone, in which he found the concrete in the hottest part of the chimney opposite the flue perfectly sound and exceptionally hard.

Concrete then may be considered as satisfactory for a lining or an inner shell in ordinary cases, although when exceptionally high temperatures are expected, say above 750° F., it is on the side of safety to employ firebrick.

PRINCIPLES OF DESIGN.

In designing a chimney, the thickness of the outside shell must be sufficient to bear, with the steel imbedded in it, the pressure due to the weight of the chimney and to the wind. A proper number of vertical steel rods or bars must be inserted all around the chimney to resist the pull caused by wind pressure, and steel hoops must also be placed at intervals to stiffen the vertical steel and prevent cracks due to difference in temperature between the inside and the exterior, and especially to resist the vertical shear which corresponds to the horizontal shear in a beam.

The computation of stresses is by no means a simple calculation because of the combined action of the concrete and steel, which involves the elastic theories. A method given in the 1906 Handbook of the Expanded Metal & Corrugated Bar Co., although somewhat cumbersome in reviewing a chimney already built, offers as good a solution as has yet been given. In computing the stresses, allowance must be made for the weight of the chimney, and the vertical shearing stress must not be neglected.

As stated above, it is customary in chimney design to build an inner shell with an air space of 3 or 4 ins. between it and the outer shell. This inner shell is sometimes carried clear to the top of the chimney, and in other cases about one-third the distance to the top, the object being both to protect the exterior shell from the high heat of the boilers and provide an insulation which will prevent the lowering of the temperature of the gases which affect the draft. Concrete chimneys frequently crack at and above the top of the inner shell, so that there is some doubt as to whether this should not be extended higher than the usual one-third. It ought to be possible, however, to lay and reinforce the concrete so as to prevent cracking even with the extremes of outside and inside heat, so that it does not seem advisable at this time to recommend a radical change in the height requirements.

The inner shell must be entirely independent of the outer, as otherwise the expansion caused by the interior heat will tend to lift and crack the chimney at any point of contact. As already stated, reports from many users of concrete chimneys have shown interior shells of reinforced concrete to be in good condition, so that concrete inner shells should be satisfactory under ordinary conditions. It may be advisable for the present, however, to limit the use of a concrete lining to a temperature of 750° F., since heat tests of concrete above this limit have not yet clearly established the effects of continued high temperature. This temperature is not often exceeded in the gases from an ordinary battery of boilers. If for any reason excessively high temperatures are to be met, the lining should be of firebrick, and special studies be made to determine how high it should be carried.

If the inner shell does not extend to the top, the portion of the outer shell at the level where it stops should be especially reinforced by an additional amount of vertical and horizontal reinforcement, as the change in temperature is liable to start cracks at this point.

Variation in thickness should be avoided if possible, and if necessary should be provided for by additional reinforcement placed near the outer surface. Changes in section are bad because stresses are introduced which are indeterminate.

Since many chimneys have developed long vertical cracks, it follows that the horizontal or circular reinforcement should be increased. Closer spacing of the reinforcing rods is also advised so as to give more of a network which will bind the concrete and prevent the first formation of cracks. A spacing of 12 ins. is certainly none too close in the lower shell.

The foundation for the chimney must be designed according to recognized engineering principles.

UNIT STRESSES.—It is good practice to compute a chimney for a wind pressure of 50 lbs. per sq. ft., reduced to 33 lbs. per sq. ft. of diametral projection to allow for the reduced pressure on the curved surface. This represents maximum velocity, and as there is no danger of exceeding it, at first thought it might seem permissible to employ a smaller factor of safety than is common in other constructions. That is, it would seem that the safe stresses might be taken nearer to the maximum computed stresses.

However, while it is true that the velocity of 100 mi. per hr. will never be exceeded, it is for several reasons unsafe to take any greater risk in chimney construction than in other engineering structures. For, if a chimney does fall, it endangers life and property; its height above ground makes inspection of the workmanship more difficult, and the attainment of perfect homogeneity less easy; and the wind causes vibrations which produce repetition of stresses. The last reason is of such importance that the discussion of the other two may be neglected. Repeated blows or repeated compressions and pulls upon any material tend to weaken it. Many tests and experiments show that reinforced concrete is admirably adapted to sustain shocks and repeated loadings much better, for example, than brickwork; nevertheless, these same tests also show that when such conditions of repeated stresses occur, reinforced concrete, like any other materials (even iron and steel), will fail at a lower stress than it otherwise would, and therefore must have a large factor of safety.

The writer would suggest the following unit values as safe for reinforced-concrete chimney construction:

Concrete, extreme fiber stress in compression, provided it is capable of attaining a strength of 2,500 lbs. per sq. in. in 28 days	600 lbs. per sq. in.
Steel in tension	14,000 lbs. per sq. in.
Steel in compression	14,000 lbs. per sq. in.
Concrete in shear	60 lbs. per sq. in.

MATERIALS.

In so important a construction as a chimney, it is necessary that all the materials entering into it shall be of very best quality.

The cement should be such as will satisfy the standard requirements of the American Society for Testing Materials, and the cement actually entering into the chimney should be carefully sampled and tested.

Sand is not usually tested in concrete work, but it should be tested in the case of any important structure. In a number of concrete failures (not failures of chimneys) which have recently come to the personal knowledge of the writer, the inferior quality of the sand has seriously contributed to the failure. If the sand appears clean and its mechanical analysis or sieve test shows that not less than 30% will pass through a sieve having 40 meshes per linear inch, and if a sand from the same bank has been used in other concrete work which has hardened satisfactorily, it may be accepted without further test, although even then it is advisable to have samples made up in the proportions and with the cement to be used, and tested in a laboratory. If the sand is dirty, or is finer than this, that is, if more than 30% passes a sieve with 40 meshes per inch, a representative sample should be sent to a laboratory and thoroughly tested, and either a coarser sand substituted or richer cement proportions than usual adopted. If the sand comes from a bank which has never been used for concrete, it should be submitted to laboratory test, notwithstanding its apparent good quality, since it is absolutely impossible to accurately pass upon any sand sample by merely looking at it.

It has been customary in the past to use a sand mortar for chimneys instead of a true concrete. There is no reason, however, why a clean fine gravel or a fine hard crushed stone, such as trap, cannot be used with the sand for the outer shell with a reduction in cost, and an increase in density which will make the concrete stronger and less porous. Instead, for example, of using a mortar 1 part cement to 3 parts sand, a concrete of 1 part cement, 2 parts sand and 2 parts half-inch stone or gravel will undoubtedly be stronger, denser and more watertight. Several chimneys have already been built using broken stone as a part of the aggregate. Coarse stone—¾-in. or 1-in.—could be introduced, and still have a strong mixture, but as this is probably less fire-resisting than a finer material it is not advisable.

Tests would probably indicate that the true concrete is suitable for the inner shell as well as the outer, but until this is proved it is perhaps advisable to continue the use of a mortar for the inner shell.

Water used in mixing is customarily required in specification to be free from acids, alkalies and vegetable matter. In chimney construction it is equally imperative that more of it be used than has heretofore been customary, so as to produce a dense mixture with perfect adhesion to the steel.

Round and shaped steel bars, such as twisted or corrugated or T-shaped, have been used in chimney construction. Steel with deformed surface gives greater adhesion than the round, but as the round is quite generally adopted in other reinforced-concrete construction, it seems quite safe to use for chimney construction. With T-shaped steel, a perfect bond with the concrete is more difficult because it is hard to get the tamping iron into the angles. Ordinary market steel whose breaking strength in tension is about 60,000 lbs. per sq. in. may be safely used. High-carbon steel would give a greater factor of safety, but unless of very high quality, which is difficult to attain without such severe specifications that the cost becomes too large for small structures, high-carbon steel is apt to be more brittle and therefore of no advantage. Even with the very best high-carbon steel, it is not advisable to allow a pull much greater than 14,000 lbs. per sq. in., because when this is exceeded there is a possibility of cracks forming in the concrete.

It is, of course, impracticable to make the steel rods long enough to reach the entire height of the chimney. They must, therefore, be lapped, and this lap should be a distance corresponding to at least 50 times the diameter.

CONSTRUCTION.

The principles of reinforced-concrete construction which have proved essential in other classes of work must be followed for chimneys. A disregard of these principles is bound to cause trouble.

First and most essential is the proper proportioning and mixing of the concrete. The measurement of the sand for every bag of cement must be made in an exact measure, and the mixing must be thorough—at least three turnings dry and three turnings wet being required. Special care must be taken in gaging the water. It is absolutely wrong, however, to lay any concrete, in which steel is imbedded to take the pull, with what is termed a very "dry" mixture. This dry consistency has been frequently used in chimney construction in the past, and appears to be a principal cause of the actual failures and to contribute to a large number of the cracks which have developed in the chimneys now standing.

The consistency with which concrete is placed must be wet enough to quake, that is, to form a jelly-like mass; not necessarily wet enough to flow, but with sufficient water so as to insure that in every batch of concrete light tamping will bring the water to the surface. When a very dry mix is used, the water may come to the surface next to the forms, and thus give a good-looking concrete, but it will be dry in the center and around the steel where it is most needed. Furthermore, by mixing just on the danger limit, occasional batches will be even dryer than the average so as to permit soft porous places in the concrete. These statements are not theoretical, but are the result of actual and extended observations of the results coming from this practice.

With a wet mix the forms cannot be raised quite so easily as with a dry mix, and the surface will not be of so uniform a color; but, nevertheless, a wet mix is necessary to produce a dense watertight concrete and to make it adhere to the steel. It has been found possible under ordinary weather conditions with a quaking mixture to raise the forms or molds the day after the concrete is placed, but to be on the safe side two sets of forms may be employed and raised alternately.

Each batch of concrete should be incorporated with the one below it, and on beginning a day's work the surface should be thoroughly cleaned and soaked with water, and then a layer of neat cement spread upon it to form a bond between the old and the fresh concrete.

It is essential in the construction that the forms be held absolutely rigid while the concrete is being placed and until after it has set. A slight disturbance may form a crack in the interior of the concrete next to the steel so as to scale off later. The steel must be carefully placed, the vertical steel being preferably not far from the center of the wall and the horizontal steel outside of it. In places where special temperature stresses occur, the horizontal steel should be as near as practicable to the stressed surface, although not nearer than 1½ ins. The steel must be well bonded, and the concrete carefully placed and tamped around it, being sure that the concrete next to the steel is actually wet and quaking.

The outside surface must be formed by the molds, no exterior plastering being permissible, as this is almost sure to check and scale off with the action of the weather.

The concrete must be laid before the initial set of the cement because regaging retards the set.

SUMMARY OF ESSENTIAL REQUIREMENTS.

(1) Design the foundations according to the best engineering practice.

(2) Compute the dimensions and reinforcement in the chimney with conservative units of stress, providing a factor of safety in the concrete of not less than 4 or 5.

(3) Provide enough vertical steel to take all of the pull without exceeding 14,000 lbs., or at most 16,000 lbs., per sq. in.

(4) Provide enough horizontal, or circular, steel to take the vertical shear and to resist the tendency to expansion due to the interior heat.

(5) Distribute the horizontal steel by numerous small rods in preference to larger rods spaced farther apart.

(6) Specially reinforce sections where the thickness of the chimney is changed or which are liable to marked changes of temperature.

(7) Select first-class materials and thoroughly test them before and during the progress of the work.

(8) Mix the concrete thoroughly and provide enough water to produce a quaking concrete.

(9) Bond the layers of concrete together.

(10) Accurately place the steel.

(11) Place the concrete around the steel carefully, ramming it so thoroughly that it will slush against the steel and adhere at every point.

(12) Keep the forms rigid.

The fulfilment of these requirements will increase the cost of the structure, but if the recommendations are followed, there should be no difficulty in erecting concrete chimneys which will give thorough satisfaction and will endure.

STUDIES OF SEWAGE DISTRIBUTORS FOR TRICKLING FILTERS.*

By C.-E. A. WINSLOW, EARLE B. PHELPS, C. F. STORY and H. C. McRAE.†

1. THE GENERAL PROBLEM OF SEWAGE DISTRIBUTION.

The chief technical difficulty in the construction of the trickling, or percolating filter, as it is called in England, lies in securing efficient distribution of the sewage. The liquid must be discharged slowly and evenly over the whole surface of the bed and thoroughly aerated in the process; in other words, it must be distributed in a rain of fine drops. The mechanism adopted differs widely in various disposal plants. Some of the earlier English investigators used tipping buckets and troughs, placed at intervals over the filter, relying on the splash to dose the intervening areas. This method must necessarily produce imperfect results, and the required apparatus is cumbrous and expensive. Other devices are designed to secure a steady overflow from fixed level troughs or pipes. Of these, the most highly developed is the Stoddart distributor, in use at Horfield and other English towns. This is practically a series of channels over the sides of which the sewage overflows continuously, dripping from points on the under side, 360 points being allowed to a square yard. Theoretically this should secure a very even distribution; but the system requires that the channels remain perfectly level and clean, a condition difficult to preserve in practice.

Another series of distributors secures an even spread of the sewage by the use of some form of moving apparatus. The simplest devices of this type consist of a pair of revolving arms discharging sewage in opposite tangential directions, so that they are propelled by their own discharge on the principle of a Barker mill. The Candy-Whittaker patents cover some of the best known English distributors of this type. More elaborate designs involve the use of power for moving troughs from which the sewage trickles by numerous small openings, or from which it flows in a thin stream over a weir. In connection with the experiments carried out at Birmingham, Scott-Moncrieff installed a large distributor of this pattern, which consisted of a trough revolving on a circular rail at its outer extremity and driven by a small oil engine. At Hanley is a similar trough, which runs back and forth over a rectangular bed, propelled by an electric motor.

There are certain inherent objections to all such movable distributors which have so far seemed almost insuperable to American engineers. Power, or its equivalent hydraulic head, is necessarily involved in all of them. Movable arms and troughs involve structural details which are expensive to construct and which require constant attention; and it may seriously be doubted whether devices of such complexity and delicacy of adjustment could be operated at all under the influence of a severe winter climate. Even in England trouble was experienced at certain plants during the cold winter of 1906-07.

The simplest and most practical of the English methods of distribution is the system of spraying the sewage upward from sprinkler heads arranged at intervals along fixed pipes. Fixed sprinklers of this type have been used for some time at Salford and other English towns, and are planned for the two largest trickling filter

*Contribution from the Sanitary Research Laboratory and Sewage Experiment Station of the Massachusetts Institute of Technology. Condensed from "The Technology Quarterly," Sept., 1907.

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plants yet designed in the United States—at Columbus, O., and Waterbury, Conn.

The chance of offensive odors is of course increased by the distribution of sewage spray in the air, but the free oxidation probably facilitates purification. At Birmingham a careful comparison has been made of various types of revolving distributors and pressure nozzles. Mr. J. D. Watson reports (Eng. News, Vol. 58, p. 89), as a result of these studies, that spray jets cost $2,500 per acre to install, against $5,000 to $20,000 for moving distributors, and that the beds equipped with the latter were idle as a result of the distributor being out of order from 8 to 28% of the time. The results obtained

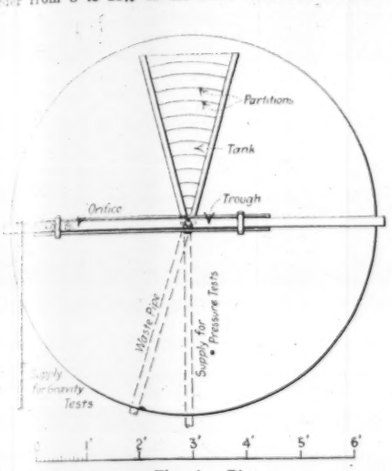


Fig. 1. Plan.

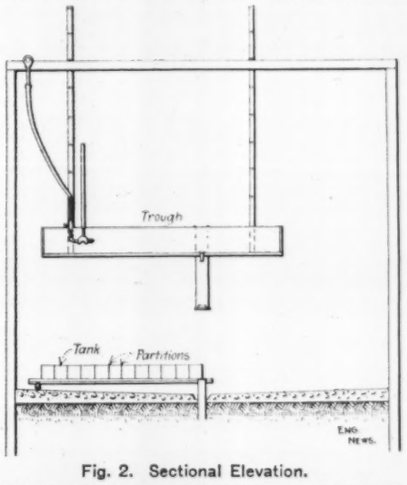


Fig. 2. Sectional Elevation.

FIGS. 1 AND 2. PLAN AND SECTIONAL ELEVATION OF APPARATUS FOR SEWAGE SPRINKLER TESTS, MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

with fixed sprinkler heads of course depend largely on the details of their construction, the head available, the spacing of the nozzles, and particularly on the construction of the nozzles themselves.

In the course of the investigations on the purification of the sewage of Boston, carried out at the Sanitary Research Laboratory of the Massachusetts Institute of Technology, another method, that of gravity distribution, has been devised.

This system, referred to in an address before the Society of Arts, Oct. 12, 1905, and described in Engineering News for Aug. 16, 1906, appears to possess certain distinct advantages. The sewage is distributed in troughs elevated over the filter, from which it drops through holes in the bottom of the troughs onto concave disks below, splashing upward from the disks in a fine spray. It has been used for two years with success at the Boston station, and has since been adopted by the Massachusetts State Board of Health for an experimental filter at Andover, according to an account published in Engineering News of April 11, 1907.

With distributors of almost any type it is possible to combine siphon tanks or other automatic devices for securing intermittency of operation. The increased pressure thus produced at the moment of maximum discharge tends to keep clear any openings which might otherwise clog, and when the sewage is discharged under pressure from perforated pipes the variations in head tend to make the circle of spray both wider and more even. This plan has long been used at Chesterfield and other English towns.

II. METHODS FOR MEASURING THE EFFICIENCY OF SEWAGE DISTRIBUTORS.

The Stoddart distributor and the various types of movable sprinklers used in England effect a theoretically perfect distribution, and their failure, when it occurs, is due to structural imperfections or lack of care in maintenance. On the other hand, fixed sprinkler nozzles and the gravity system of distribution cover at best only a portion of the surface of the filter. Since both these latter types discharge sewage approximately in a circle, each nozzle or disk will wet only 78% of the square area surrounding it. Furthermore, almost all these distributors tend to discharge, for the most part, in one or two concentric rings of the wetted area. Some nozzles, too, discharge unevenly along different radii. It is important to determine how serious these deviations are. Imperfect distribution means that a part of the filter area below is wasted and another part is operated at excessive rates. Yet almost no exact data upon this point are now available. Some experimenters have noted the proportion of the total area wetted by their sprinklers; but such a standard is, of course, wholly inadequate, since it fails to discriminate the extent of the discharge upon different portions of the wetted area. We have therefore thought it worth while to make a somewhat extensive study of this point, comparing in detail the efficiency of various fixed sprinkler nozzles and of a number of patterns of gravity distributors under various conditions.

For the quantitative study of the efficiency of distribution a measuring apparatus was devised, the construction of which is indicated in Figs. 1 and 2. It consisted of a shallow, circular basin of cement concrete, 14 ft. in diameter, sloping toward a central drain connected with a waste pipe below. A 2-in. supply pipe passed under the concrete and rose through this drain at the center of the basin. To this pipe were attached the sprinkler nozzles of various types designed for pressure distribution. A wooden platform in the shape of the 30° sector of a circle was attached at its pointed end to a 2½-in. iron collar, fitted over the supply pipe so as to turn freely upon it. The outer end of the wooden platform was supported on ball-bearing castors, arranged to run on a flat shelf at the periphery of the concrete basin, so that the whole platform could be revolved about the supply pipe as a center; and the collar and castors were so adjusted that the platform was exactly level. The platform carried a galvanized iron tank, also in the form of a 30° sector, divided into compartments by concentric arcs 6 ins. apart. The radius of the tank was 6 ft., giving twelve concentric compartments, and its depth was 6 ins. When this tank was rotated about the supply pipe, during the course of an experiment, the evenness of distribution was of course measured by the relative amount of liquid collected in the respective compartments. In various stationary positions the trough recorded radial inequalities.

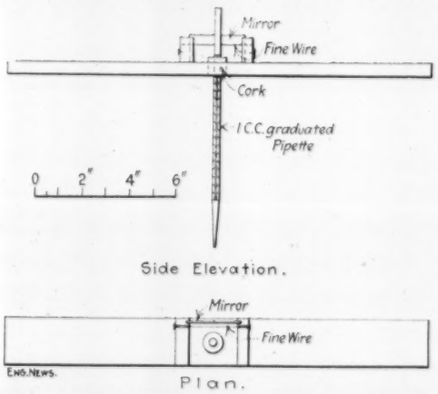


Fig. 3. Plan and Elevation of Gage for Measuring Depth of Sewage in Sprinkler Testing Apparatus.

The supply pipe at the center of the basin was connected at its outer end with an upright stack of 2-in. pipe made up of short lengths, any desired number of which could be used. At the top of the stack was a wooden box, fed by a hose and relieved by an overflow just below the top. The difference between the water level in this box (varied by removing sections of pipe from the upright stack) and the level of the sprinkler nozzle at the center of the basin represented approximately the total head effective in distribution. The rate of flow was of course fixed by the head and the sprinkler nozzle used. It was estimated in each test from the total discharge into the measuring tank, and from the time occupied by the run.

The apparatus, as shown in Figs. 1 and 2, was adapted for the study of gravity distributors by the addition of an overhead trough. This was hung from two timbers passing diametrically across the basin, supported at their extremities by upright studs. At a point fixed over the center of the apparatus a ¾-in. brass orifice in the bottom of the trough allowed the liquid to discharge upon the splashing disk below.

The uprights supporting the trough were fixed to the cross-timbers above by movable iron pins and the uprights were pierced for the reception of these pins at intervals of 1 ft., so that the distance between the trough and the measuring tank below could be varied at will. The distance between the level of the water in the trough and the top of the tank measured the total head available for gravity distribution. In this system, however, there is another variable. With a given total head, the distance between the point of discharge and the splashing disk and the distance from the splashing disk to the filter may be mutually varied. In our experiments this was provided for by supporting the various splashing disks tested upon a circular lead plate, 6.5 ins. in diameter, hung from the distributing trough by four wires. Varying the length of these wires altered the portion of the total head above the splashing disk (called for convenience "head-on-sprinkler").

The rate of flow with this type of distributor is, within limits, independent of the head, so that the rate also had to be taken into account. The rate of discharge was equal to the rate of supply of liquid to the trough, and this supply was measured through a calibrated orifice, indicated in Figs. 1 and 2. Two orifices were used in the experiments, one $^{7}/_{16}$-in. and one $^{5}/_{8}$-in. in diameter. The orifice plug in use was connected to a 2-in. tee by a reducing coupling. The section of the tee was large enough to permit an accurate measurement of the static head on the orifice by a glass piezometer tube inserted in its top, and the head was adjusted at any desired point by a valve behind the tee.

The orifices were calibrated by measuring their discharge at various heads in a cypress tank, 2 x 6 ft. in area and 3 ft. deep. A special form of gage, to be described later, was used in these measurements, and they were accurate within .005-in. in the depth of the tank, or .005-cu. ft. of discharge.

The measurement of the volume of liquid collected in the various compartments of the measuring tank during a test was, according to our original plans, to be made directly, and for this purpose a small stopcock was attached to each compartment. This process proved somewhat tedious, however, and the same end was attained by measuring the depth of the liquid in each compartment at the end of the run, after a careful preliminary calibration of the various compartments.

The depth of water in this measuring tank and in the cypress tank used for nozzle calibration, was measured by a simple and accurate gage devised for the purpose, the construction of which is indicated in Fig. 3. It consisted of a graduated glass tube (a 1 c.c. pipette), the graduated portion being 6 ins. long and divided into hundredths. This tube slid up and down in a cork fitted into a long, flat piece of wood, carefully planed. When the wooden crosspiece rested in a horizontal position, supported at its ends, the tube between was exactly vertical. Just above the wooden crosspiece a fine wire was stretched longitudinally between two supports behind the tube, and behind the wire a small mirror was fixed. By reading with the wire and its reflection superposed the position of the tube could be estimated to tenths of its smallest graduations (.0005-ft.). In measuring the depth of water in a compartment of the measuring tank the wooden crosspiece of the gage was rested on the two sides of the tank and the tube was slid carefully down till its mouth just touched the surface of the water. This point could be detected with great accuracy by the jump into the tube of the drop of water drawn up by capillary action.

The testing apparatus in operation is shown in Fig. 4. The following memorandum of the routine procedure in making the test of a gravity distributor may make the working of the whole apparatus somewhat clearer. In beginning a run the measuring tank was emptied, put in place on the wooden platform, and covered with a cover of tar paper stretched over slats. The desired total head was adjusted by raising or lowering the trough, and the head on the sprinkler was regulated by means of the wires suspending the lead plate from the trough. The disk to be tested was then centered on the lead plate just below the ¾-in. opening in the bottom of the trough. The valve supplying the trough was then opened and regulated till the desired discharge was registered on the piezometer tube. After conditions had become constant in the trough, the cover of the measuring tank was removed (the time being taken with a stop watch). During the test the tank was slowly rotated by hand. As soon as any compartments became so full as to threaten splashing the test was stopped, the cover replaced, and the time again noted. The supply valve was then closed, and after the discharge had ceased the cover was removed and the depth of liquid in each compartment of the measuring tank was determined.

III. METHOD OF CALCULATING A COEFFICIENT FOR THE EFFICIENCY OF SEWAGE DISTRIBUTORS.

After the proportionate amount of discharge in various zones about the distributors had been determined by the method outlined above, it was necessary to devise some method of stating the results so that the efficiency of the various types could be easily and accurately compared. For this purpose one of us (E. B. P.) has derived a "coefficient of distribution," which shows the mathematical relation between any observed condition of distribution and that ideal condition in which each small unit of area should receive its exact proportionate share. Professor Phelps' calculations were published in Engineering News of Oct. 18, 1906, and in the Technology Quarterly for March, 1907 (Vol. 20, p. 34).

It has been found simpler and sufficiently accurate in practice to make an arithmetical calculation of the coefficient directly from the observed data.

Either the total excessive discharge on the areas receiving too much liquid, or the total deficient discharge on the areas receiving too little, when divided by the total discharge measures the deviation of the distribution from a perfect condition. [And this percentage of deviation, deducted from unity, or perfect distribution, gives the percentage of efficiency of the sprinkler, which in this discussion is called the crude coefficient, since there remains to be considered the unwetted area between the circular wetted area and the circumscribing square. Obviously, if sprinklers are so spaced that the circular areas wetted by each do not touch a still greater correction must be made; and if they overlap it still becomes necessary to correct the crude coefficient.—Ed.]

In practice it would be of advantage to arrange distributors, not along two axes at right angles to each other, so that each is at the center of a square, but on three axes inclined at 120°, so that each is at the center of a hexagon. With distributors at the centers of squares, the waste area, with no overlapping, amounts to 21.5% of the whole. With distributors at the centers of hexagons, the waste area amounts to only 9.9%. The hexagonal arrangement makes it possible to put 343 sprinkler heads on an acre instead of 303, spaced with 12 ft. c to c., and to raise a rate of 2,000,000 gals. to 2,270,000 without increasing the discharge upon any portion of the wetted area.

IV. DETERMINATION OF COEFFICIENTS FOR GRAVITY DISTRIBUTORS.

The system of gravity distribution involves the discharge of the sewage from openings in the bottom of troughs onto concave disks placed below, the discharged stream having sufficient force to splash up and out from the disks in a fine spray.

The splashing disks first used on the Technology trickling filter in 1905 were of metal, 3 ins. in diameter, and with a concavity corresponding to a radius of curvature of 6 ins. These were at first placed only 5 ft. apart, with a discharge on each disk of less than 9 gals. per min. This discharge proved too low for good distribution, and a single disk was substituted for four on each 100 sq. ft. in area, giving 10 ft. between disks and a discharge on each of 3.47 gals. per min. This proved far more satisfactory, and the experimental filter thus equipped gave very satisfactory results even through the severe winter of 1906-07.

With gravity distributors there are three distinct variable factors, the rate of discharge, the distance between the trough and the disk, and the distance between the disk and the filter below. In our experiments we have varied all three independently. For rates we have used the three standard discharges described above, corresponding, respectively, to 1.71, 4.08 and 8.47 gals. per min. For the total head between the trough and the measuring tank (corresponding to the filter surface) three standard heads were used, 2 ft., 4 ft., and 6 ft.; and with each of these total heads three different positions of the disk were tested. With the 2-ft. total head the head-on-sprinkler was made 6 ins., 12 ins. and 18 ins.; with the 4-ft. total head the head-on-sprinkler was made, 1, 2, and 3 ft.; with the 6-ft. total head the disk was placed at 1, 2½ and 4 ft.

Besides these three variable factors the efficiency of gravity distribution depends upon the design of the splashing disks themselves. Twenty-three types of disks were tested in our experiments, varying in size and curvature, in the hope of finding some more favorable design than the simple concave type first used. Cross-sections of the disks examined, with their respective dimensions, are show in Fig. 5.

Nos. 1 to 4 were wooden disks 2 ins. in diameter, with surfaces of different degrees of concavity varying from a radius of curvature of 4 ins. to one of 20 ins. Nos. 5 to 10 were disks 3 ins. in diameter, Nos. 6 and 7 of metal and the rest of wood, the radii of curvature of their concave surfaces varying from 2 to 20 ins. Nos. 11, 12, and 16 were larger, very shallow wooden disks. Nos. 13, 14, 17, 18, and 19 were wooden disks whose upper surfaces were concave about a central raised point, rising again all around their outer edge. We have called this design the mushroom disk. No. 15 was a large, deep metal disk, 5 ins. in diameter, and with a 3-in. radius of curvature. Finally, Nos. 20, 21, 22, and 23 were large, 6-in. wooden disks with radii of curvature varying from 4 to 20 ins.

Since each of these 23 disks was tested under 27 standard conditions (three subdivisions of each of three total heads at three rates), making over six hundred tests in all, it has not seemed best to publish the mass of figures obtained in detail. The final corrected coefficients are given in three tables [omitted here.—Ed.], arranged first according to the rate of discharge on each disk and then according to the head.

It must be remembered, of course, that the corrected coefficients in these tables take account both of the extent and the evenness of the circle of spray. Some of the disks produced good distribution within a small circle, but the rate on the wetted area was so great as to be excessive. By reducing the crude coefficients for such disks so as to bring the total rate down to 2,000,000 gals. per acre per day, the corrected coefficients have been estimated. This was done for the three rates of discharge tested as follows: A discharge of 1.71 gals. per min. on each splashing disk corresponds to 2,480 gals. per day per disk. At a 2,000,000-rate this requires 808 sprinklers per acre, or one sprinkler to each 54 sq. ft. of filter area; the distance between sprinklers must therefore be 7.35 ft. Similarly, the 4.08 gal. rate would require 341 sprinklers per acre, or one to each 128 sq. ft., and the distance between sprinklers must be 11.3 ft. The 8.47 gal. rate calls for 164 sprinklers per acre, or one to 266 sq. ft., with a distance of 16.3 ft. between sprinklers. All coefficients in the tables have

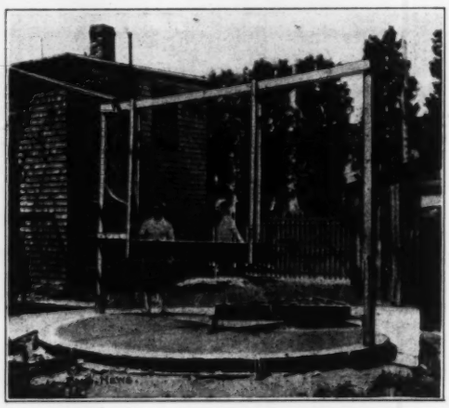


Fig. 4. View of Apparatus for Testing Sewage Sprinklers.

therefore been corrected by referring the actual discharges to squares, respectively 7.35, 11.3 and 16.3 ft. on a side.

Taking up the independent variables tested in their order it is at once apparent that the efficiency of gravity distributors is markedly influenced by the rate of discharge on each disk, and that about 4 gals. per min. is the most favorable rate. The lower rate, 1.7 gals. per min., gave very poor results with the larger disks, the liquid merely dripping over the edge. With the smaller disks fairly good results were obtained within a radius of 3 or 4 ft. of the disk, although there was generally an excess of discharge nearest the center. As the distance between sprinklers for this head was only 7.35 ft., the crude coefficients of .6 and .7 were not so greatly reduced as in other cases, and for the smallest (2-in.) disks this rate is better than either of the higher ones. Most of the 2-in. disks gave corrected coefficients of over .4 even with the 6-ft. total head, and coefficients of over .5 were obtained for disks 4, 6, and 10.

The intermediate rate of 4.08 gals. per disk per min. did not suit the small, 2-in. disks as well as the lower rate, but with larger disks it gave better results. With good heads the liquid was well distributed, showing neither the concentration near the disk obtained with the lowest rate nor the marked outer ring characteristic of the higher one. Crude coefficients of over .7 were obtained with almost all the disks by using a 6-ft. head, and the better ones ranged between .8 and .9. Furthermore, the wetted area was proportionately larger than with the higher or the lower rate of discharge; 4.08 gals. per disk per min. calls for a distance of 11.3 ft. between sprinklers, and with high heads the better disks wetted circles 10 and 11 ft. in diameter. The corrected coefficients remained, therefore, relatively high, several of the disks showing values over .5 and one, disk No. 7, over .6.

With the highest rate the better disks covered a 10-ft. circle, but there was usually a ring of excessive discharge at a distance of 3 or 4 ft. from the center. The larger disks, at high heads, gave crude coefficients of between .7 and .8, wetting a circle of 6-ft. radius. Corrected for a 16.3-ft. square these crude coefficients are seriously reduced.

The head used with the sprinkler disks was the most important factor in controlling their efficiency. The 2-ft. total head proved inadequate in most cases. Some of the crude coefficients were fairly good (between .6 and .8); but the wetted area was always much too small, giving an excessive rate near the sprinkler, and the corrected coefficients were almost always under .1. The 4-ft. total head showed better results, and the 6-ft. better still, the improved efficiency being directly proportional to the increase in head.

With regard to the best proportion of the total head to be used in the discharge above the disk, results are somewhat more variable. Increasing the head above the disk gives an enlargement of the wetted area, but it apparently brings about also an increased tendency to concentrated discharge in a single zone.

There remains still to be considered the influence of the shape of the splashing disk itself. Two general patterns were studied—simple concave plates and disks of the "mushroom type" with a raised central cone (Nos. 13, 14, 17, 18, and 19). The latter show coefficients intermediate between the large and small concave disks, better than the former and not so good as the latter. In other words, the slope on one side of the central boss deflects the water very much as a simple cup of similar size and curvature would do. The discharge from these disks was, however, unequal along different radii, since it was almost impossible to make the stream of liquid strike the central boss. The coefficients are therefore higher than they should be, since the revolving trough caught portions of unequal discharges in different zones along the various radii. There seems to be no advantage in the use of disks of this pattern.

Efficiency evidently increases with the increasing concavity of the disks. Disks Nos. 12 and 16, with very flat surfaces, gave such poor results that they have not been plotted. The improvement with increase in curvature is most marked with the disks of small diameter, and with the 6-in. disks the 12-in. radius of curvature is the maximum, the 4-in. radius giving very poor results. An increase in concavity increases the efficiency of the disk by breaking the liquid up more completely and sending it higher into the air. The accompanying increase of disk surface over which the liquid must travel tends, however, to decrease the efficiency of distribution, probably by the greater friction involved. With small disks this is not serious, but as the surface is increased this factor limits the possible increase in concavity. Disk No. 15, with a diameter of 5 ins. and a radius of curvature of 3 ins., represents an extreme condition in which the liquid did not splash at all, but eddied about in the deep cup and merely dripped over the edge.

The only other point noted was the effect of the composition of the disks. Nos. 6, 7, and 15 were of metal, spun out of No. 18 sheet copper. All the others were of white pine, turned and carefully smoothed on the lathe, and varnished. No. 15, as noted above, was of very poor shape, but Nos. 6 and 7 gave the best results obtained in the whole series. No. 7 had a 2-in. radius of curvature, and should therefore have given better results than the others, which were of less concavity. No. 6, however, was of the same size and shape as No. 10 (of wood). Its coefficients were distinctly better under the most favorable heads. Thus at the 4.08-gal. rate, with a 6-ft. total head and 2.5-ft. head-on-sprinkler, No. 6 gave .49 against .42 for No. 10; with 4-ft. head-on-sprinkler it gave .56 against .44.

In general, it appears from this portion of the investigation that the best results with gravity distribution can be obtained under the following conditions:

(1) The discharge on each sprinkler should be in the neighborhood of 4 gals. per min.; this means, for a 2,000,000-gal. rate, 340 sprinklers per acre, with a distance between the sprinklers of about 11 ft.

(2) The head between the distributing trough and the filter should be as great as possible; 2 ft. is inadequate, 4 ft. gives fair results, and 6 ft. is better. Where exceptional conditions make still higher heads possible, the use of large, deep disks might prove of advantage.

(3) The head on the sprinkler should be from 2 to 4 ft. The best subdivision of available total head can probably best be determined by experiments with the disks to be used in each individual case.

(4) A simple concave disk of metal seems to promise the best efficiency.

(5) The best diameter for the disks appears to be 3 ins. For low rates of discharge smaller disks are better, and for very high rates or very high heads larger ones may be more suitable.

(6) Unless the disk be too large it is of advantage to increase its concavity as much as possible. Of 3-in. disks, that having a curvature corresponding to a radius of 2 ins. proved most satisfactory. The radius of curvature might profitably be increased toward the limiting value of 1½ ins., which would make the disk a hemisphere. With larger disks larger radii of curvature are necessary.

Of all the disks studied, the best was No. 7, a metal

...3 ins. in diameter, with a radius of curvature of ... With a total head of 4 ft. and a head-on-sprinkler ... ft., it gave a coefficient of .40, the best obtained ... that total head; and with a total head of 6 ft. and ... head-on-sprinkler of 4 ft., it gave the highest coefficient ... for any gravity distributor, namely, .62.

DETERMINATION OF COEFFICIENTS FOR PRESSURE DISTRIBUTORS.

The system of spraying sewage upward from fixed sprinkler nozzles under pressure was developed more or less independently at various plants in England. At Salford it was worked out after the failure of various other methods, in which the sewage overflowed from troughs, or was spread by a thin layer of sand covering the filter proper. In the earlier experiments with fixed nozzles, a disk-like cap was placed a little above each in order to break the sewage into a fine spray. Then the attempt was made to break up the stream by the impact of two converging flows; and finally a nozzle was designed which discharged the sewage from a series of openings arranged spirally about a center. In Derbyshire, Barwise describes as common the use of fixed perforated pipes with metal disks placed over the outlet to form a spray (very much like the earliest Salford type). At Birmingham small sprinkler heads are used, in which the sewage passes up through a narrow, annular opening and is broken up and thrown outward by striking the bottom of a metal plug raised a little above it.

In America the first tests of trickling filter distribution were made at Columbus, O. For the preliminary experiments of 1904-05 a nozzle was designed which recalls that used at Salford. The sewage was discharged through eight ⅛-in. brass pipes, at an angle of 45° with the vertical, into the space between an inner and an outer cone. The sewage left the nozzle in a tangential direction and spread in a fine sheet. Clogging proved too serious an objection to this nozzle, and another design was finally adopted. The latter was described by Mr. J. H. Gregory (Trans. Am. Soc. C. E., Vol. 57, p. 130) as follows:

> The nozzle, which is of brass, consists of a single orifice, $^9/_{16}$-in. in diameter, with rounded edges, above which, held by two thin arms, is an inverted 90° cone, the axis of the cone coinciding with the axis of the orifice. The jet, on leaving the orifice, impinges against the cone and is transformed into a thin sheet, spreading out radially and breaking into a shower of fine drops. At the points where the sheet strikes the arms the latter are ground down to sharp edges on each side, thereby permitting the sheet to heal up entirely after passing them.

At the Waterbury Experiment Station, Mr. W. G. Taylor has made a careful study of the general operation of sprinkler nozzles, and has devised still another type. He reports (Eng. Record, Vol. 55, p. 10) that a single opening, with a cone above for distribution, supported at the sides as in the Columbus nozzle, proved unsatisfactory. The side supports broke the sheet badly, especially after the development upon them of a fungus growth. Furthermore, he points out that the uniformity of distribution obtained by means of a single cone acting under a head of 5 ft. was not satisfactory. Although the efficiency as marked by the area wetted was approximately 75%, a certain outer zone invariably received sewage at a higher rate than the inner area. To lessen this defect, and to secure a greater percentage of wetted area and a more uniform distribution of the sewage, the idea occurred to the writer of placing a lower and secondary cone over the orifice, with the object of intercepting a portion of the sewage and distributing it over the inner fifth of the circular area within the range of the nozzle. The result was effected by boring a hole through the lower cone slightly less in diameter than the fixed orifice in the nozzle, the diameter of the opening in the cone being so proportioned to the diameter of the nozzle orifice that four-fifths of the rising stream passes through the lower cone to be diffused by the upper, and the remaining fifth is sheared off by the lower cone and sprinkled over the central area.

It is evidently desirable to have an accurate measure of the comparative performance of these sprinkler nozzles, and of the still better designs which the future will probably develop. Mr. Gregory, in describing the Columbus experiments, was content to say that the nozzle used wetted 77% of the tributary area "in a perfectly satisfactory manner." Mr. Taylor noted, but did not measure, serious inequalities of distribution within the wetted area. We were therefore particularly glad to apply our method of measuring efficiency to the most promising of these pressure nozzles, and the results have proved significant. Through the kindness of Mr. G. C. Whipple, of New York, we have been furnished with two of the nozzles used at Salford and with the Birmingham type. To Mr. Gregory we owe the Columbus sprinkler, and Mr. Taylor has courteously given us drawings of the Waterbury design. The tests of these five types may best be discussed one by one, and reviewed in comparison with each other later.

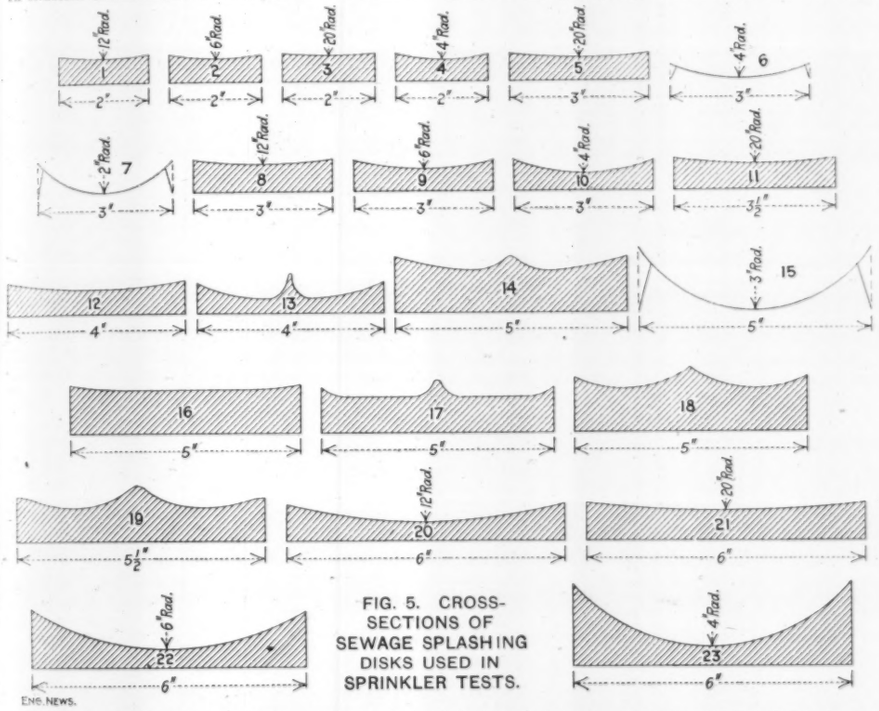


FIG. 5. CROSS-SECTIONS OF SEWAGE SPLASHING DISKS USED IN SPRINKLER TESTS.

The Salford nozzle (old style) is a brass nozzle in the form of a cup, with two openings in the sides at such an angle that the streams discharged upward from them meet in the cup itself. Its construction is shown in Fig. 5.

This nozzle differed from all others tested in the fact that it did not discharge equally along different radii. It threw the liquid up in a fan-shaped spray, in quite a narrow plane. Our ordinary method of rating efficiency, which assumes symmetrical distribution about the center, was of course inapplicable under such circumstances. Instead, therefore, of revolving the measuring tank about this sprinkler, we measured the discharge at six different stationary positions, immediately adjacent to each other and together covering one-half of the entire circle surrounding the distributor. The average discharge in each zone was used in the manner described above for calculating the crude coefficient. Then the mean discharge along each sector was determined, and from this the total excessive discharge along those sectors which received more than their share. The ratio of the excessive radial discharge to the total discharge was then determined and taken from unity, giving the "radial coefficient." The product of the crude coefficient and the radial coefficient measures the evenness of distribution within the wetted area.

When this sprinkler nozzle was tested under a head of 6.4 ft., another anomaly appeared in connection with the determination of the corrected coefficient. The area wetted was larger than the square which would be tributary to each sprinkler at a 2,000,000-gal. rate. This means, of course, that at this head and rate the circles would overlap. The following method of calculation was applied in this case and in others of the same kind.

The effect of overlapping circles of spray is exactly the same as it would be if each portion of the spray passing the boundary of the tributary square were perfectly reflected at the side of that square and returned to a corresponding point within. It is necessary, therefore, to calculate how much of the discharge into the outer rings would have been thus reflected, or, in other words, what proportion of each of the overlapping rings falls without the square. Given the dimensions of the square and of each ring, this is a simple problem in trigonometry, the solution of which gives,

$$\frac{P}{100} = \frac{\cos^{-1}\frac{S}{D}}{45^\circ},$$

in which S is the side of the square and P the percentage of the total discharge into any overlapping ring of diameter D, which would be reflected to an inner ring. Having subtracted from all the rings the amounts which would be reflected, it is next necessary to distribute these amounts among the inner rings where they properly belong. The mathematical solution of this problem gave a result too cumbrous to use. A graphical representation of the conditions drawn on plotting paper, however, indicated readily and with sufficient accuracy the amounts of the reflected discharge which would fall into each of the inner rings affected and the reflected water was then redistributed. These new values were then recalculated like the others and a coefficient obtained, which referred, not to a circle, but to the area common to the maximum circle and the intersecting square. The relation between this area and that of the square furnished the required factor for the calculation of the corrected coefficient from the crude.

The results of the tests on the Salford old style nozzle are shown in [a table here omitted, as are the other detailed tables. A comparative summary is given near the close of the paper.—Ed.]. The total head, in this and succeeding tables, is the difference in level between the surface of the liquid in the supply box and the top of the measuring tank. The head-on-sprinkler is the difference in level between the surface of the liquid and the nozzle itself. In general, the nozzle was placed as close as possible to the top of the tank.

One very interesting point about this nozzle is brought out in the column of rates. The discharge with a head-on-sprinkler of 6.2 ft. was less than with a head-on-sprinkler of 3.9 ft. This is apparently due to the fact that the two converging streams discharged by this nozzle meet in its cup with such force as to produce a back pressure, which at high heads cuts down the flow appreciably.

This nozzle secures a fairly even distribution within the wetted sector, improving with the head to a very high value. There is an area of excessive discharge near the outer limit of the wetted sector, but the excess is not great. The inequality of distribution along different radii is much more serious. Nearly half the liquid is discharged in excess on one of the six 30° sectors, and this weakness causes the products and corrected coefficients to be poor.

The Salford new style sprinkler tested is a double cone, the outer cone being open at the top. The liquid enters the space between the cones through six holes of $^5/_{32}$-in. diameter, inclined at an angle to the major axis, so that a spiral motion is produced within the inner cone. This sprinkler is shown in Fig. 6.

With this, as with the other pressure nozzles, distribution along different radii was approximately equal.

The results of these tests indicate that the new Salford sprinkler gives very good results at the higher heads. The tests themselves show an excessive discharge on the outer part of the wetted area, but the excess is small.

The Birmingham nozzle, shown also in Fig. 6, discharges through a ⅝-in. opening. Through this opening passes a $^7/_{32}$-in. shank, the enlarged upper portion of which breaks up the ascending ring of liquid and sends it out in a fine spray. In this case the discharge through the opening was so small and the distribution so good that a circle larger than the proper tributary square was wetted in every test except those made at the 2.2-ft. head.

[This test is] of the greatest interest as showing what results may be obtained with a nozzle of small bore so arranged as to break up the stream quickly and completely. It is clear that the Birmingham nozzle effects a far more perfect distribution than any other device we have examined.

The Columbus nozzle is designed on a plan exactly opposite to that of the Birmingham sprinkler, in so far as it aims to enlarge the orifice of discharge as much as possible. The Columbus design has been described above, and is also shown in Fig. 6.

The Columbus sprinkler shows at all heads a fair

distribution within the wetted area, its departure from the ideal condition being due to the formation of a zone of excessive discharge a little inside the periphery of the wetted area. Crude coefficients, however, are always about .6, and one notable thing about this nozzle is the fact that it gives fairly even distribution even at the 2-ft. head. The wetted area is, however, small in proportion to the discharge, and therefore the corrected coefficients are poor. Thus at the 4.3-ft. head, for example, the rate of discharge of the nozzle was 10.9 gals. per min. This discharge, if the total rate on the filter be 2,000,000 gals. per acre, calls for 121 sprinklers per acre, which means a distance of 19 ft. between sprinklers. The wetted circle at this head was only 6 ft. in radius, so that at an ostensible 2,000,000-gal. rate half the filter area would be working at 4,000,000 gals. and the rest would be idle. It is no doubt for this reason that the Columbus engineers plan to operate at a 4,000,000-gal. rate for half the time, and to rest the whole filter the remainder of the time, spacing their nozzles 16.2 ft. apart. This would of course give a distribution more nearly approaching our crude coefficients, though the results at best would be below .6.

The Waterbury nozzle used in these experiments was of the type described on a preceding page, in the citation from Mr. Taylor's paper. Its construction is indicated in Fig. 6. Briefly, the nozzle consists of two cones, one above the other, the lower one perforated at its center. An annular discharge is in part deflected outward by the lower cone, while its inner portion passes through the lower cone and is deflected by the upper one. The result is to produce two concentric cones of spray and thus improve the distribution obtained with a nozzle of the Columbus type. In the sprinkler tested by us, the position of each cone in relation to the point at which the liquid leaves the pipe below could be varied by means of set screws, and a number of different settings of these cones were examined. The distance between the lower opening and the lower surface of the lower cone at its outer rim is designated by the letter a; the distance between the upper surface of the lower cone and the lower surface of the upper cone is designated by the letter b.

It will be noticed that the rates [shown by the tests] varied somewhat under the same conditions. This was no doubt due to the tendency of this sprinkler to clog. Our tests were all made with water, but the tanks through which it was pumped were not perfectly clean and at times contributed their quota of suspended solids (old sewage sludge). These inequalities did not, however, seriously affect the coefficients, which are fairly uniform. It appears from this table that the relative position of the cones does not greatly modify the efficiency of distribution.

The wetted area with this nozzle is small for its high rate of discharge.

Mr. Taylor has worked out certain modifications of his sprinkler which have not been examined in the present investigation, but which well deserve a careful study. Two of these are described in the paper previously cited (Eng. Record, Vol. 55, p. 10) and more fully in a somewhat later one (Eng. News, Vol. 57, p. 238).

VI. THE EFFICIENCY OF PRESSURE DISTRIBUTORS WITH VARYING HEADS.

With a sprinkler which, like many pressure nozzles, discharges in a conical sheet and forms an excessive discharge on a peripheral zone, it would seem to be of advantage to vary the head-on-sprinkler in such a way as to cause the zone of excessive discharge to travel in and out over the wetted area. This may easily be done by the interposition of siphon tanks in the supply system, and such is the practice at many of the English plants, notably in Derbyshire (Barwise, "Purification of Sewage," London, 1904). The Board of Advisory Engineers to the Baltimore Sewerage Commission have suggested a similar arrangement for their projected trickling filters (Report of the Board of Advisory Engineers and of the Chief Engineer, Baltimore, 1906). At Columbus, Mr. Gregory, in the paper previously cited (Trans. Am. Soc. C. E., Vol. 57, p. 130), states that the sprinklers will be operated at a head varying during 24-hour periods from 2.75 ft. (a rate of 10 gals. per min. per sprinkler) to 5 ft. (a rate of 13.5 gals.). We have had a Columbus nozzle with varying head in operation on the trickling filter at the Technology Experiment Station since the summer of 1906 with apparently good results, and the Massachusetts State Board of Health has used a siphon tank in connection with their gravity distributor at Andover (Eng. News, Vol. 57, p. 397).

It is clear that such varying heads must tend to improve the efficiency of devices like the Columbus sprinkler. Just how much improvement is effected under such conditions can, however, only be determined by exact measurement. We have therefore made a study of the operation of the Columbus sprinkler at varying heads, and the results have proved of considerable interest.

We used in our series of tests a maximum head of 4 ft. (in two cases 3.7 ft.) and a varying minimum head from 3.5 down to 2 ft. The discharge was made from a square siphon tank equipped with float valves so as to start and cut off sharply.

It was expected, a priori, that a varying head would give more even distribution within the wetted area; and to the eye the waxing and waning cone of spray looks well-nigh perfect. Nothing could better illustrate the importance of the rigorous method of measurement adopted in these experiments. The crude coefficients are distinctly poorer with varying than with constant heads. An inspection of the original measurements shows the presence of a well-marked zone of excessive discharge just as in the case of the constant head, only not so far out from the center of the wetted area. A large excess of the liquid was found in the compartments 2 and 2½ ft. out from the nozzle. The smaller radial distance of this maximum zone accounts for its greater effect in reducing the coefficient.

On the other hand, the corrected coefficients were much improved by the intermittent operation. The wetted area was as great as in the tests at a constant head of 4 ft. and the discharge was much less. The average rate was as low as 4 gals. per min. per sprinkler. Therefore, for a total rate of 2,000,000 gals. per acre the nozzles could be placed much closer than at a 4-ft constant head (11 ft. instead of 19), and the wetted area made to approximate more nearly to the tributary squares. Thus the corrected coefficients for a head varying from 4 ft. down range from .3 to .4 against a little over .2 for the constant head of 4.3 ft.

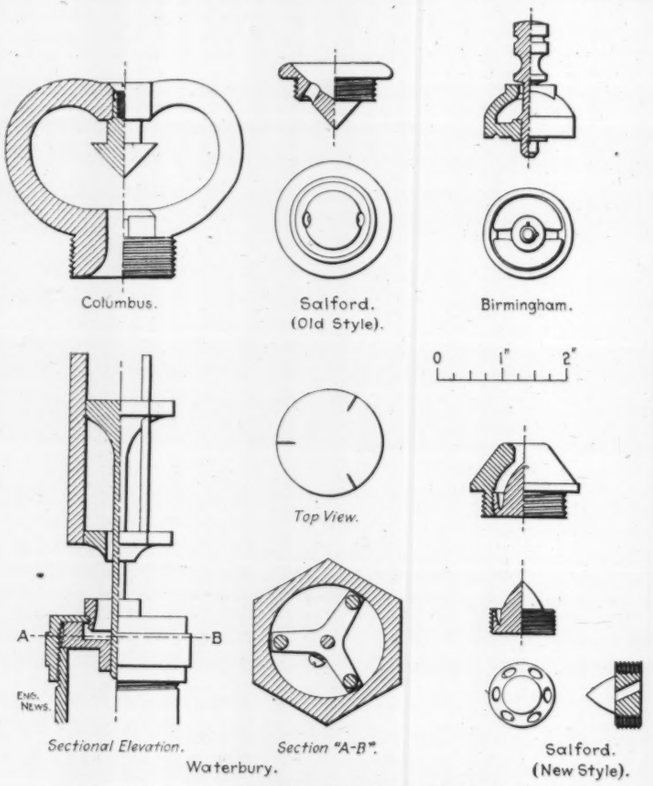


FIG. 6. TYPES OF PRESSURE NOZZLES USED IN SEWAGE SPRINKLER TESTS.

VII. COMPARATIVE EFFICIENCY OF DISTRIBUTORS OF VARIOUS TYPES.

The optimum conditions for distributors of various types have been somewhat fully discussed, but a brief comparative summary of their performance may be of interest in closing. We have therefore summarized the results under optimum conditions (4-ft. total head for Birmingham, 6-ft. for the others) in [the accompanying table.]

Comparative Efficiency of Sewage Distributors of Various Types.

Type.	Rate, gals. per min.	Coefficient. Crude.	Coefficient. Corrected.	Sprinklers per acre.*
Best gravity	4.08	.76	.62	341
Old Salford	2.09	.44	.41	483
New Salford	2.1	.78	.67	667
Birmingham	2.0	.80	.80	700
Columbus	14.8	.61	.30	94
Waterbury	10.4	.73	.22	134

*To discharge 2,000,000 gals. per acre per day.

The American pressure nozzles, judged on the basis of these measurements, show a low efficiency in distribution. They cover the wetted area fairly well under the best conditions, but their discharge is so excessive that at a total rate of 2,000,000 gals. per acre they leave large waste areas between, and their corrected coefficients are correspondingly low. It must, of course, be remembered that at a rate of 4,000,000 gals. per acre, for which the Columbus nozzle, at least, is designed, the corrected coefficients would be much higher. Furthermore, the performance of these nozzles may be further improved by the use of varying head, which increases a coefficient of .2 (for 4-ft. total head) to .4 (for a head varying between 3.5 ft. and 4 ft.). This question of varying heads well deserves further study, especially in the light of Mr. Taylor's plan for parabolic orifices. The Columbus nozzle offers great advantages in the simplicity of its construction and in its large orifice, if its excessive discharge can be overcome by the use of siphon tanks.

The best of the gravity distributors (a 3-in. metal disk with a 2-in. radius of curvature) has given much better results than we had hoped for it. Without the complication of siphon tanks and varying head, it shows a corrected coefficient of .62, double that obtained for the Waterbury and Columbus nozzles under the same conditions. With a 4-ft. total head its coefficient is .40, while the Columbus nozzle gives .22 with a similar constant head, and the best Columbus nozzle value with a varying head is only .43. The smallest opening for clogging in the gravity system is the discharge pipe, ¾-in. in diameter, while the diameter of the Columbus nozzle is 9/16-in. Furthermore, the accessibility of the whole distribution system makes its maintenance in good condition easy.

The new Salford and Birmingham pressure nozzles produce the most efficient distribution of any of the devices tested. At a 6-ft. total head the Birmingham coefficient is .69, and at a 4-ft. total head it is still better (.80), the distribution being almost perfect. These good results are, however, gained by the use of nozzles which are extremely liable to clogging. The openings of the new Salford nozzle are only 3/32-in. in diameter, and the cone into which they discharge is admirably adapted to collect solid materials. The opening in the Birmingham nozzle is a ring only 2/64-in. in diameter. Its removable plug indicates that even with the carefully screened and septicized sewage of Birmingham it requires frequent attention; and it is difficult to see how any such device could be economically used under American conditions, even with setpic sewage. On the other hand, crude sewage can be sprayed by the Columbus nozzle or the gravity distributor without difficulty from clogging.

STEEL BAND DRIVING BELTS are produced by the Eloesser Kraftband Gesellschaft, Charlottenburg, Germany, according to a note in the "Zeitschrift des Vereins Deutscher Ingenieure." These belts are narrower than equivalent leather belts, can be used successfully on very short spacing of driver and driven pulley, can be run at higher speeds, and have no stretch. The band of steel is butt-jointed with a short inside and a long outside splice-plate, joined through the band with a single row of screws or rivets on each side of the joint. The inner splice-plate is curved to the radius of the smaller pulley. The long ends of the outer splice-plate flare away from the band slightly, serving as guards to prevent any sharp bending at the joint. The pulleys for these steel belts may be smooth or covered with special friction covering. The apparent slip is stated to be only about 0.1%. Several quoted examples are: The main drive of a 200-HP. steam engine, 150 r.p.m., pulleys 148 ins. and 32 ins. in diameter, is a steel band 4 ins. wide by 0.02-in. thick, velocity 5,300 ft. per min. The belt from a 125-HP. gas engine running at 186 r. p. m., pulleys 127 ins. and 36 ins., is 4 ins. by 0.02-in. and has a velocity of 6,200 ft. per min. The drive from a 60-HP. gas engine of 180 r. p. m., pulleys 110 ins. and 24 ins., is 2¾ ins. by 0.012 in. and has a peripheral velocity of 5,200 ft. per min. The driving belt of a 50-HP. traction engine, 110 r. p. m., pulleys 55 ins. and 46 ins., is 4 ins. × 0.024-in. and has a peripheral velocity of 1,600 ft. per min.

CONSTRUCTION AND UNIT COSTS OF CONCRETE LOCK, ROUGH RIVER, KENTUCKY.*

(Contributed.)

Rough River is a tributary of Green River, emptying into it at a point 71 miles from its mouth at the Ohio River, near Evansville, Ind. The lock herein described is situated on the Rough River, near Hartford, Ohio Co., Ky. Its general dimensions are: c. to c. of quoins, 145.75 ft.; width between walls, 27.0 ft.; length of lock walls, 180.25 ft.; height, 24.0 ft.; width, 12.0 ft. at base and 5.0 ft. at the top, except at the ends and opposite miter sills, where it is 10 ft. The lock has a lift of 9.0 ft.

The excavation for the lock walls was moved in the usual manner, partly by dredge and partly by dump cars. The river side was in water from 5 to 6½ ft. The lock pit was enclosed on the river side and ends by an ordinary double-walled cofferdam, puddled between walls, and on the outside with material excavated from the lock pit. The enclosure within the cofferdam was about 60 ft. wide and 209 ft. long. When completed the water at the deepest place in the pit was 6 ft. 7½ ins. With a centrifugal pump having a 10-in. suction it required on the first trial about 2 hrs. and 30 min. to remove the water to inlet of suction, or within 1 ft. 4 ins. of the bottom of the pit. After completing the pumping, the bottom of the pit was cleaned of mud, which exposed almost a level ledge of sandstone for a foundation base for the lock walls.

The abutment on the opposite side of the river, against which the end of the dam rested, was well inland and required the excavation of a pit about 26 ft. deep to the same ledge referred to above. The excavation was removed from the abutment pit partly with wheelbarrows and partly with a derrick. On this ledge the frames for both lock wall and abutment were erected. The plan of frames for the lock wall is shown by Fig. 2. It will be noticed that the vertical post on face of wall, the batter post at back, the two top ties and the strut at each end of the ties form a bent when put together. These bents were spaced 4 ft. apart and formed the essential parts of the concrete forms.

In erecting the timber in place, after being framed in the yard, it was run out on a car to the end of the approach trestle or end of the erected frame, taken from the car by a derrick, and laid in the bottom of the pit. The posts, ties and struts were then bolted together, and the bent in this form was raised and set in place by a derrick.

It was first made fast at the top by two iron dogs, with one end made fast to the last bent set up and the other ends secured to the new bent. The bent was then made plumb and braced in place on both sides by the struts at the bottom and the middle braces for the posts. In this manner all the posts and ties were put together and erected. The completion of about ten bents was a day's work of 8 hrs. It required about 5½ days to erect all the bents from end to end of lock wall.

In determining the proportion of the ingredients for the concrete, careful experiments were made to determine the voids in the sand and in the various mixtures of the aggregate. From the data thus obtained it was determined to make the facing one part cement to two parts sand and the heart of the wall of a mixture of one part of Germania Portland cement, two parts of Ohio River washed sand, two parts of washed gravel, and three parts of broken Green River limestone, which passed a screen with 2-in. meshes and was cleaned of all dust or other pulverized matter.

As the river wall has two exposed faces, both were composed of a 6-in. layer of mortar, as mentioned for the facing above. The top of the lock wall was finished with about 3 ins. of this facing, and the top of the abutment with a mixture of one part cement, two parts sand and three parts gravel.

Of the 2,959 bbls. of cement used in the lock wall and abutment during the first season, 2,056 bbls. were used in the concrete and 903 bbls. in the mortar facing. From the experience gained it was found that 2 or 3 ins. of this facing was sufficient for all practical purposes and in later construction 3½ ins. was used instead of 6 ins.

Before commencing the work it was concluded that a perfect condition of the stone, gravel and sand would be to have them so full of water that they would not absorb any more when put in the mixer, but such a condition, as is well known, can only be approximately obtained. The practice was to keep these ingredients as wet on the yard as was possible by having a 1-in. hose play on them for about four hours each day. Such watering commenced about five days before the first mixing of concrete, and was continued until the concrete was all in place.

The ingredients of the concrete measured separately were for one batch of about 1 cu. yd., 4.7 cu. ft. of cement, 9.5 cu. ft. of sand, 9.5 cu. ft. of gravel, 14.1 cu. ft. of stone, and from 2.21 cu. ft. (16½ gals.) to 2.74 cu. ft. (20½ gals.) of water. The above are the extreme quantities of water used. The amount of water was governed by the absorption of the aggregate, and as that could not be made uniform the measure of water was varied from hour to hour as the condition of the concrete being rammed in place indicated. The rule for water was to use sufficient only to have it show on top of each layer after ramming. It was aimed to secure this condition and at the same time avoid quaking; however, quaking sometimes resulted, and occasionally there was insufficient water to show on top after ramming. The mixer was a cubical box of 45 cu. ft. capacity. The wall was divided into nine sections, ranging from 18 to 24 ft. in length, alternate sections being filled. The progress of the work was governed by the tampers. It was practicable to work only 8 to 13 men tamping in a section from 18 to 24 ft. long and from 5 ft. wide at the top to 12 ft. at the bottom. The progress vertically was from 15 to 20 ins. per hr., and it required from 13 to 22 hrs. to put in sections varying from 140 to 240 cu. yds.

The usual force for concrete work was from 48 to 54 men, including foreman, engineer and carpenters. However, on one section, which was placed during a rain, both in day and night, the force was reduced to 35 men.

In mixing, the cubical mixer was revolved nine times on work done the first season. The next

*Compiled from the Annual Report of the Chief of Engineers, U. S. Army.

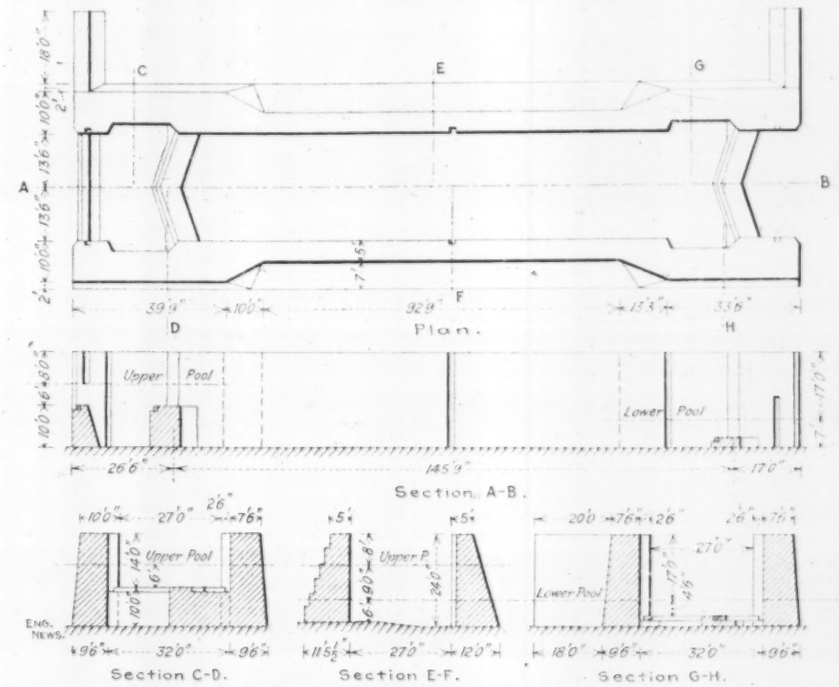


FIG. 1. CONCRETE LOCK, ROUGH RIVER, KY.

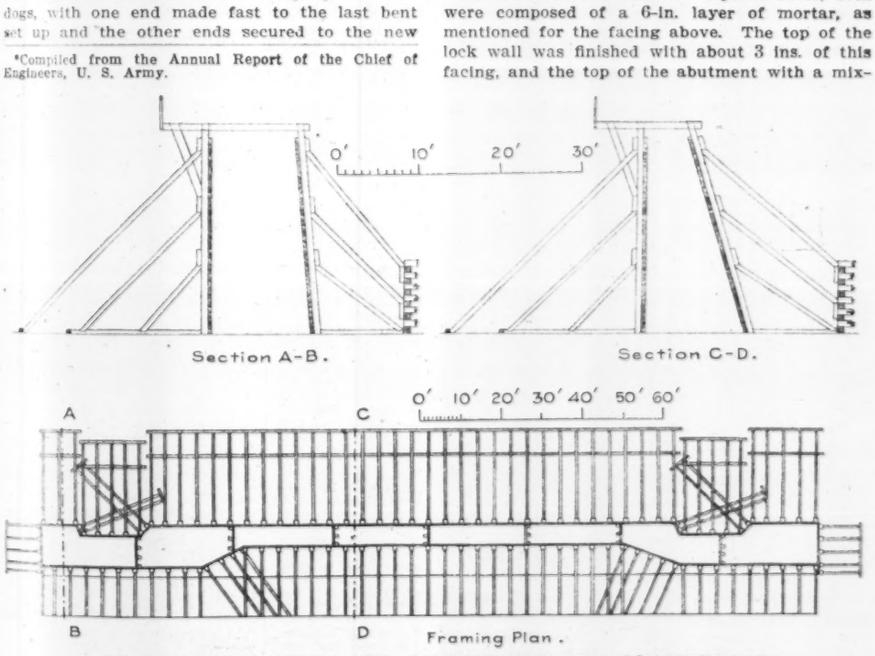


FIG. 2. FRAMING DETAILS FOR CONSTRUCTION OF A CONCRETE LOCK.

season this was increased to from 15 to 18 revolutions. As soon as a section was finished it was covered with canvas and kept covered until well hardened. The finished work was sprinkled with water. The sheeting was removed after five days.

The following statement shows the total and unit costs of lock and dam, all the work on which was done by hired labor under direct employ of the government engineers.

Items—Classes of Work.	Amounts.	Cost per unit.
Excavation:		
Lockwalls and abutment:		
Above water.....10,774 cu. yds.		
Below water..... 6,668 cu. yds.		
17,422 cu. yds.	$5,030.02	$0.29
Channel below lock:		
Clay ..... 3,463 cu. yds.		
Stone ..... 6 cu. yds.		
3,469 cu. yds.	761.38	0.22
Lower approach to lock 2,610 cu. yds.	253.33	0.10
Upper approach to lock 2,794 cu. yds.	450.45	0.16
Grading lock grounds:		
Fill back of land walls 2,031 cu. yds.		
Grounds ..... 460 cu. yds.		
2,491 cu. yds.	905.33	0.36
Grading about abutment	297.75	....
Removal of old lock, river wall, etc..... 1,500 cu. yds.	324.85	0.22
Excavation of dam.... 3,800 cu. yds.	861.00	0.23
Total	$8,884.11	
Puddle:		
Cofferdam ..... 2,147 cu. yds.	$223.57	$0.10
Dam ..... 1,718 cu. yds.	343.00	0.20
3,865 cu. yds.		
Total	$566.57	
Timber.		
Cofferdam:		
Material .....98,000 ft. B. M.	$1,129.72	$11.53
Labor, framing and placing.....	650.00	6.63
Dam:		
Material .....164,530 ft. B. M.	2,485.30	15.10
Labor, framing and erecting.....	1,583.00	9.61
Guide cribs:		
Material .....66,349 ft. B. M.	1,053.64	15.88
Labor, erecting	1,570.80	23.67
Breakwater:		
Material .....12,000 ft. B. M.	19.56	1.63
Labor	80.80	6.73
Gates (4 in all):		
Material—		
Timber .....8,434 ft. B. M.	1,321.32	150.66
Cast iron .....14,165 lbs.		
Wrought iron .....9,378 lbs.		
Labor	1,185.70	140.58
Total	$11,079.84	
Stone filling:		
Dam ..... 1,676 cu. yds.	$1,752.44	1.04
Guide cribs ..... 890 cu. yds.	625.80	.70
Breakwater ..... 169 cu. yds.	.....	....
2,735 cu. yds.		
Total	$2,378.24	
Drain pipe:		
Back of land wall..... 146 lin. ft.	$123.95	.85
Concrete:		
Materials $24,613.78		6.69
Labor mixing and placing..... $5,284.46		1.43
Materials for trestle and wharf..... 721.53		.20
Labor erecting & removing trestles.. 837.34		.23
Materials for concrete forms..... 3,334.12		.90
Labor framing concrete forms..... 1,140.00		.31
Labor erect. & removing con. forms. 3,535.56		.96
Total	$39,466.79	$10.72
Buildings:		
Materials	$1,611.62	
Labor erecting same.....	993.68	
Total	$2,605.30	
Storing and watching plant.....	$1,169.29	
Removal of plant from lock site.....	643.06	
Miscellaneous	4,221.06	
Clearing river of snags, etc., 23 miles	1,888.87	82.12
Total	$7,922.28	
Plant:		
Machinery	$7,644.05	
Small tools	1,377.59	
Total	$9,021.64	
Grand total	$82,048.72	

Recapitulation.

		Per cent.
Excavation	$8,884.11	10.8
Puddle	566.57	0.7
Timber and iron	11,079.84	13.5
Stone filling	2,378.24	2.9
Drain pipe	123.95	0.2
Concrete	39,466.79	48.1
Buildings	2,605.30	3.2
Miscellaneous	4,221.06	5.1
Watching	1,169.29	1.4
Removal of plant.....	643.06	0.8
Clearing river	1,888.87	2.3
Plant	9,021.64	11.0
	$82,048.72	100.0

Upon the completion of the work the following materials were on hand, and of the estimated value:

Material for concrete	$398.50
Material for trestle and wharf.....	75.00
Material for concrete forms.....	750.00
Material for buildings	150.00
Estimated value of plant:	
Machinery	6,000.00
Small tools	600.00
Total	$7,973.50

The total cost of the work, which included the following items, was:

Clearing river of trees, snags, etc.; removing old lock walls; lock walls and abutment of dam, built of concrete; dam, built of timber and filled with stone; guide cribs, above and below lock; gates of lock; approaches to lock; grading of lock grounds; lock employees' house; tool house, and store house..	$82,048.72
Deduct value of materials, machinery, etc.....	7,973.50
Total net cost	$74,075.22

Upon the completion of the lock and dam, all the plant, comprising machinery, tools, etc., not needed was transferred to the site of Lock No. 5, on the Green River, Kentucky, and used there, so that the amount stated above, viz., $74,075.22, represents the exact cost of the Rough River lock at the time of its completion.

As the concrete comprises the largest item of the work, a statement has been prepared showing in detail the cost of this particular class of work. The price for cement given was the cost delivered in the storehouse at the lock site. The cost f. o. b. at Livermore (the nearest railroad station) was for 2,059 bbls. at $2.88, and for 900 bbls. at $2.52. The cost of unloading from cars, transporting seven miles to work, and storing was 48½ cts. per bbl. The cost of the broken stone, gravel and sand on barge at the lock site was, respectively, $2, $1.19 and $1.04 per cu. yd. The cost of unloading and storing them in the yard was, respectively, 22, 20 and 18 cts. per cu. yd.

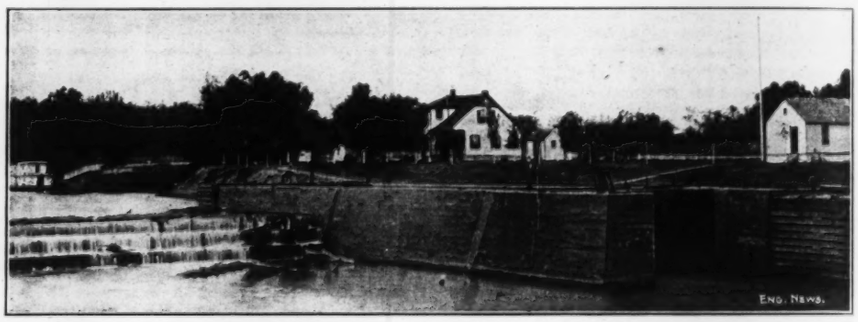


FIG. 3. CONCRETE LOCK ON ROUGH RIVER, HARTFORD, KY.

The following statement shows the itemized cost of 3,680 cu. yds. of concrete, all the work being done by hired labor:

	Amounts.	Cost per cu. yd.
5,339 bbls. cement:		
2,189 bbls., at $3.36½.....$7,365.98		
1,100 bbls., at 3.00½..... 3,305.50		
2,050 bbls., at 2.98 ..... 6,109.00		
	$16,780.48	$4.55
1,676.7 cu. yds. broken stone, at $2.22	3,722.27	1.01
1,043 cu. yds. gravel:		
719 cu. yds., at $1.39..... $999.41		
324 cu. yds., at 1.43..... 463.32		
	1,462.73	.40
1,065.2 cu. yds. sand:		
967.2 cu. yds., at $1.22....$1,179.98		
98 cu. yds., at $1.40..... 137.20		
	1,317.18	.36
Miscellaneous	932.62	.25
Total	$24,215.28	$6.57
Labor, erecting and removing forms...	3,535.56	.96
Labor, mixing and placing concrete..	5,284.46	1.43
Labor, erecting & removing approach trestles to forms	350.00	.10
20% (for each year) of cost of timber and framing same for forms....	1,789.65	.49
25% (for each year) of timber and iron for approach trestles to forms	150.00	.04
20% (for each year) of work of bldgs..	762.52	.21
15% (for each year) of cost of plant..	2,706.49	.74
50% (for each year) of cost of erecting mixer plant	124.56	.03
Miscellaneous	548.27	.15
Total	$39,466.79	$10.72

One peculiarity of the construction was that the exposed faces of the walls were made of cement mortar, varying from 3 to 6 ins. in thickness. For this reason it is difficult to calculate the "shrinkage or reduction" that took place in the finished concrete from the measured quantities of ingredients.

We find that there was used in all:

Cement	712 cu. yds.
Sand	1,065 cu. yds.
Gravel	1,043 cu. yds.
Broken stone	1,677 cu. yds.
Total	4,497 cu. yds.

This made 3,680 cu. yds. of finished concrete, showing a reduction of 4,497 — 3,680 = 817, or 18% only, which hardly expresses the true percentage. It is probable for the concrete, starting that the reduction is not far from 25%, which shows plainly that dry concrete cannot be consolidated as well as a wet mixture.

A DEFENSE OF NATIVE CHINESE ENGINEERS has recently appeared in the "Peking & Tientsin Times" in connection with an investigation of the railway from Peking to Kalgan, which is being constructed entirely with native Chinese from the Chief Engineer, Mr. Jeme-Tien-Yow, Assoc. M. Inst. C. E., down to the coolie laborers. After a portion of the road had been completed and put into service there were numerous statements made in the local and foreign press to the effect that there were no native engineers in China who were capable of carrying on any extensive or complicated railroad work. To refute this Mr. Ad. Birkhäuser, of Peking, investigated the work being done on the Kalgan road and published an account of his studies, a part of which is as follows:

Although being absolutely disinterested in Chinese railway enterprises, I was glad to seize the opportunity of a trip to the Ming tombs to stay over a few days in the Pass in order to persuade myself of the existence of that dilatory work which was not only mentioned by the writers in the public press, but prognosticated ere this by pessimistic critics on every attempt on the part of the Chinese to replace the foreigner.

Mr. Yu, engineer in charge of the place of Nankou received me very courteously, and offered me every assistance I might possibly need. The Nankou station with its extensive area covered with locomotive sheds, workshops, rail-depots, etc., did not offer much interest though it deserves recognition for the total absence of the classic Chinese disorder. It would take too long to give full details of my inspection of the works in the Pass itself, where I moved hither and thither on service trains, on donkey's back, and trolleys. Suffice to say that on my return home my notes and impressions found absolute corroboration by old foreign railway hands. I will summarize by saying that there was no trace of dilatory work existing or waste of time or money to be recorded. The question of money is naturally very elastic in things Chinese. However, I have figures to go on which I know to be trustworthy and from which I can gather with certainty that there was no waste. I would even go a point further and state that Europeans could hardly have worked so economically. The Engineer in Chief has put to advantage all his facilities, as a native, to obtain the most advantageous terms from contractors yet obtaining thorough work.

The track up to Chu-Yung-Kuan, with a gradient of 1 in 30, is thoroughly well built and the water drainage laid out in such a way as to cope easily with high mountain waters. Besides the two tunnels which were already completed this time last year, there is a third one near the last named place, 1,200 ft. long. It has been drifted from both ends simultaneously and it denotes undeniable skill on the part of the engineers when it can be said and proved that the two sides met without mistake in alinement and level. The 3,450-ft. tunnel near Pataling was begun last year and will be completed by end of January, 1908. It is also being worked from both ends, in addition to which a vertical shaft has been sunk in the middle where the abundant water met with is successfully coped with by means of cage water lifts. After completion of this tunnel the really difficult part of the whole line will be mastered, the Pass has then been overcome, and the total 125 miles to Kalgan will be ready for traffic by 1910.

The Engineer in Chief, so ably assisted by the three engineers, Messrs. Yu, in Nankou, H. K. Chang in Chu-Yung-Kuan, and T. C. Yen in Pataling, have proved that they can work without foreign assistance and that their experience enables them to work not only in flat land, but to cope with the difficulties of mountainous regions. It therefore is not uninteresting to note that they have refuted Col. Wingate's (British Intelligence Dept.) statement at a recent meeting of the Royal Geographical Society in London, "that the Chinese Engineer was not yet born that would engineer a railway through or over the Nankou Pass."

THE FLOW OF WATER THROUGH SUBMERGED TUBES: RESULTS OF EXPERIMENTS AT THE UNIVERSITY OF WISCONSIN.*

By C. B. STEWART,† Assoc. M. Am. Soc. C. E.

The recent construction of large works in connection with the various lines of hydraulic engineering has emphasized the need of further experimental knowledge concerning the losses of head resulting from various forms of resistances interposed in flowing streams of considerable size. The loss of head resulting from the flow of water through submerged gates, tubes and orifices of considerable size and with various forms of entrance, is one of the most important divisions of the general subject which needs experimental investigation. Practically the only reliable experimental data available concerning the flow of water through large orifices is given in a valuable paper by Mr. Theodore G. Ellis, entitled "Description and Results of Hydraulic Experiments with Large Apertures at Holyoke, Massachusetts."‡ Mr. Ellis determines the loss of head for flow through apertures, the dimensions of which were 2 ft. square by 1 ft. when discharging freely into the atmosphere. The paper presented also a very few experiments on the loss of head for flow through submerged apertures 1 ft. square and 1 ft. in diameter under heads from 2 ft. to 15 ft. In nearly all the experiments of Mr. Ellis, the thickness of the edge of the aperture remained constant, about ½-in. for wood and ¼-in. for iron, and perfect contractions of the emerging stream were supposed to take place on all sides of the aperture. In the single case of the aperture 1 ft. square, the entrance was subsequently modified so as to have a curved approach on all four sides, and the resulting coefficients of discharge were determined.

In the experiments at the University of Wisconsin described in the present paper, the cross-section of the orifice or tube has been kept constant, being 4 ft. square, and the effects of changing the length of the tube and of modifying the entrance conditions by curved approaches on one, two, three and four sides, respectively, for each of the various lengths, has been studied.

In future experiments on this subject at the University, it is planned to determine the coefficient of discharge for submerged tubes and orifices of various forms and areas of cross-section, with and without entrance modifications. It is hoped that the results of the experiments when completed will fairly cover the field of investigation laid out and be of material aid to the engineering profession.

WATER SUPPLY.—The experiments were made at the new hydraulic laboratory of the University of Wisconsin, located on the shore of Lake Mendota, which furnishes an abundant supply of fresh water. Exceptional facilities were afforded for experiments on a large scale, and care was taken that errors of observation should be reduced to as small a percentage as possible. Fig. 1 is a plan of the laboratory, showing the main features and arrangement for the experiments. The water was led from the lake through a suitable inlet and raised about 12 ft. by means of a 30-in. Morris centrifugal pump, and delivered into a receiving basin, whose dimensions are about 15 × 18 ft. and 10 ft. deep. From the receiving basin the water flows through a flume 10 ft. wide, 10 ft. deep and about 68 ft. long, which may be used for operating a 30-in. turbine located at the end of the flume or may be wasted through gates underneath the turbine. The water flows back to the lake through a tail race 10 ft. wide. At the outlet end of the tail race is a sharp crested weir with end contractions suppressed and having a length of crest of 10 ft. Suitable baffle boards were inserted in the receiving basin and at the entrance to the flume until it was found by trial that the water in the flume flowed in lines practically parallel to the sides of the flume.

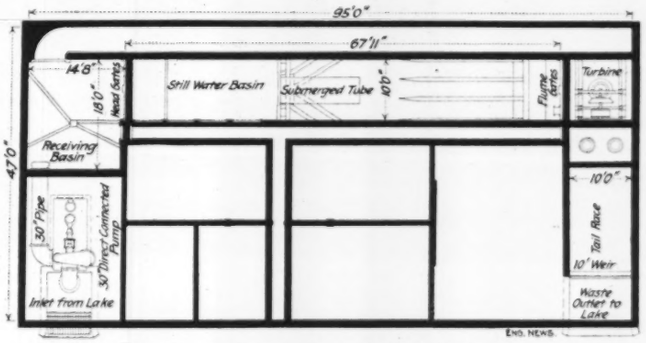


FIG. 1. PLAN OF THE HYDRAULIC LABORATORY, UNIVERSITY OF WISCONSIN.

The bulkhead containing the tube experimented with was located about 24 ft. from the entrance to the flume, the arrangement being shown in detail in Figs. 2 and 4.

WEIRS FOR THE MEASUREMENT OF WATER.—The facilities for making a volumetric calibration of the weir with a length of crest of 10 ft. were not available, so the expedient was devised of dividing the 10-ft. channel into three similar channels, each having a length of about 15 ft. and with a sharp crested weir at the end of each channel. In order to form a variable height of water surface on the downstream side of the bulkhead containing the submerged tube, the three sharp crested weirs were made adjustable in height, as shown in Figs. 2 and 3. The main features of the weirs consisted of a fixed bulkhead, extending about 5 ft. above the bottom of the channel and a movable gate above the bulkhead extending the full width of the 10-ft. channel and capable of being raised or lowered by means of two hand wheels, one on each side of the channel. The crest of the three weirs was made of a single plate of wrought iron, 6 ins. × $\frac{3}{16}$-in., bolted to the upper edge of the movable timber gate and beveled on the downstream side of the top edge to a thickness of about $\frac{1}{16}$-in. The joints around the weir openings were made water tight by means of gaskets formed of cloth-insertion, rubber tubing, ½-in. in diameter, placed horizontally at the top of the fixed bulkhead and extending vertically upwards, on each side of the weir opening, to the maximum height of water in the channel of approach. This is shown in Fig. 3. The movable gate was drawn against the gaskets by means of lag screws passing through cleats adjustable vertically in the timber frame supporting the bulkhead and gate. Each of the three weirs had its own channel of approach, about 15 ft. long. The sides of the channels were lined with 1-in. pine surfaced flooring boards so that all three of the channels of approach and weirs were practically alike, the only variation being that of a slight difference in width in the case of one weir. The board linings of the channel were extended down stream from the crest of the weir a distance of 1¼ ft. and also below the crest of the weir a distance of about 2 ft., so that the weirs had their end contractions suppressed and the filaments were guided in parallel planes after passing over the crest. The board lining around the weir crest were designed to be removed each time the crests were adjusted and were attached by countersunk flathead screws. Just above the crest the boards were 5 ft. long and extended up stream in the approach channel so as to avoid a joint on the vertical line above the crest of the weir. The slight openings between the weir-boards and the ends of the channel, below the level of the weir crest were caulked with oakum and made practically watertight, to prevent cross currents which would interfere with the linear flow over the weir crest. The arrangement of the weirs with intervening spaces of about 8 ins. between the ends, allowed free entrance of air beneath the nappes.

The measurements of heads on the crest of the weirs were made by connecting the weir channels with still water basins located below the weirs, as shown in Figs. 2 and 3, and taking readings of the elevations of water surfaces in the still-water basins by means of accurate hook gages. The connections between the weir channels and still-water basins were made by 1-in. wrought-iron pipes and 1-in. rubber tubes, as shown. The tubes were attached to the still-water basins by means of malleable iron unions and were easily disconnected. By lowering the outlet end of a tube and allowing free circulation of the water, the air in the connecting pipe and rubber tube

*Abstract of a forthcoming bulletin on the results of experiments with submerged tubes, 4 ft. square, at the University of Wisconsin, Madison, Wis.

†In charge of Research Work in Hydraulic Engineering at the University of Wisconsin, Madison, Wis.

‡Transactions of the American Society of Civil Engineers, Volume 5, 1875.

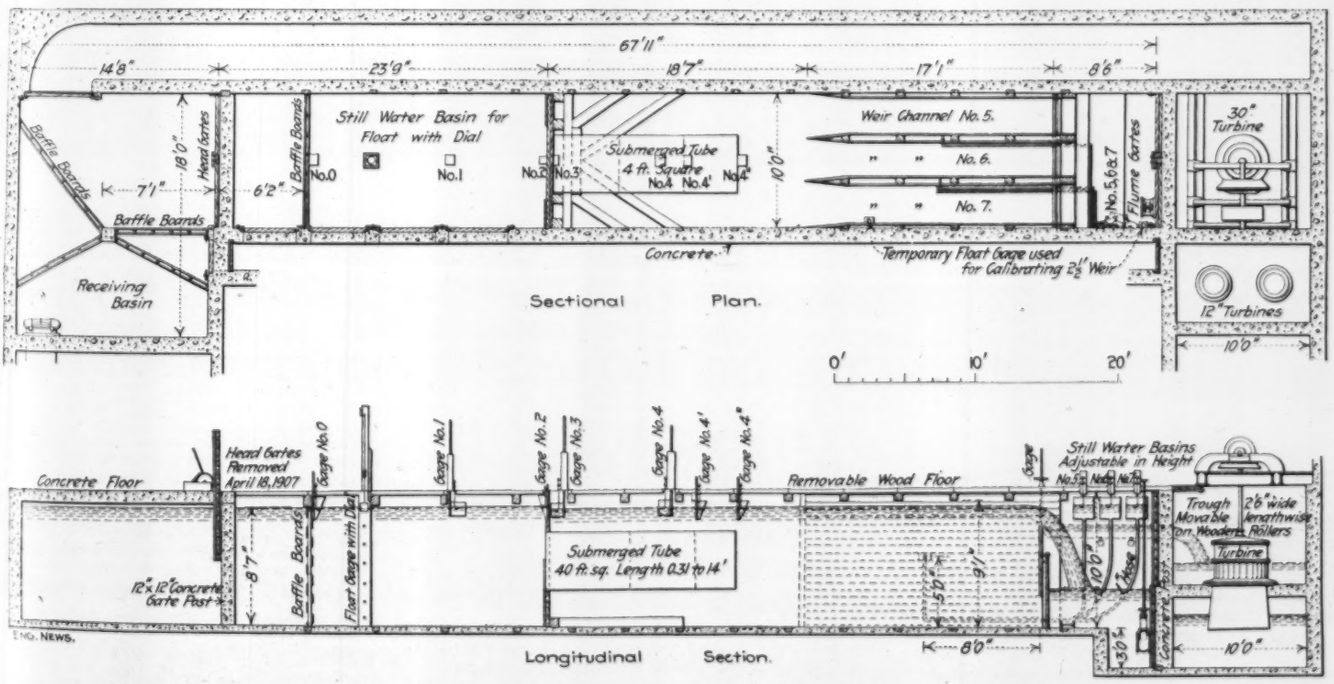


FIG. 2. PLAN AND SECTIONAL ELEVATION OF WEIR CHANNELS FOR TESTS WITH SUBMERGED TUBES.

could be easily removed. The connections with the weir channels were made at points 8 ft. upstream from the crest of the weirs, 5 ft. above the floor of the channel and about 2½ ft. below the crest of the weirs.

DESCRIPTION OF SUBMERGED TUBES.—The plan followed was to study the simple forms of submerged tubes, 4 ft. square and of length varying from $^{21}/_{100}$ to 14 ft., finding the losses of head and coefficients of discharge for the various rates of flow through the tubes. The entrance to the tubes of various lengths were then modified by introducing curved approaches, elliptical in

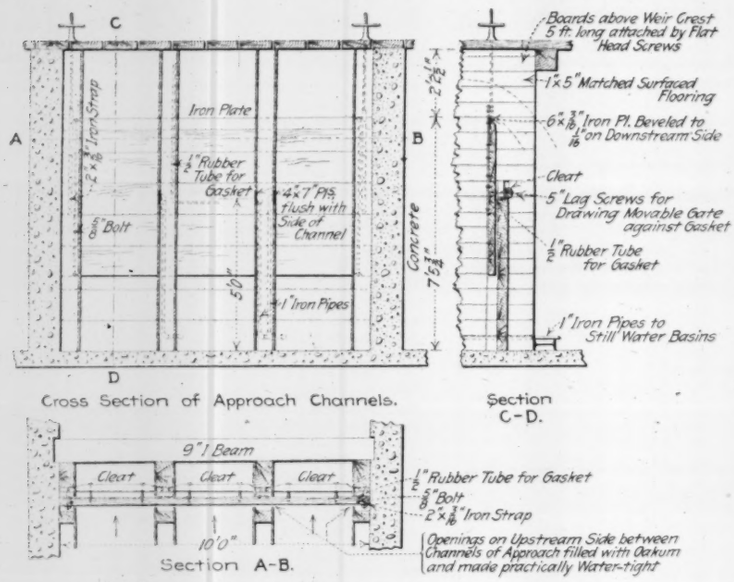


FIG. 3. CROSS-SECTIONS OF WEIR CHANNELS, SHOWING ARRANGEMENT OF WEIR CRESTS FOR EXPERIMENTS WITH SUBMERGED TUBES.

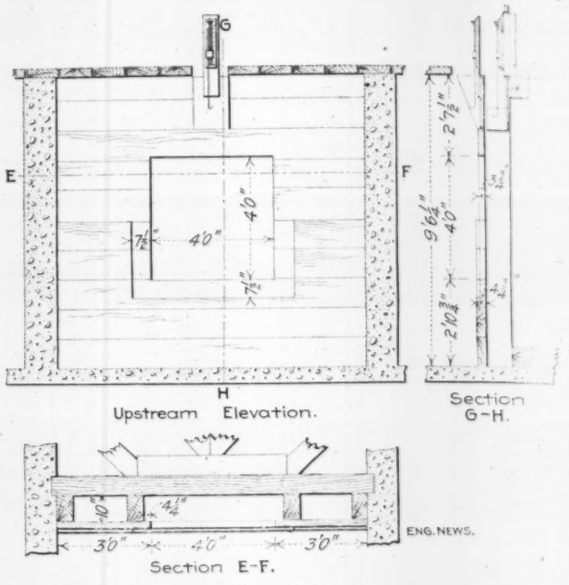


FIG. 4. DETAILS OF BULKHEAD FOR HOLDING SUBMERGED TUBES. UNIVERSITY OF WISCONSIN EXPERIMENTS.

form, thus suppressing the contraction, first, on the bottom; second, on the bottom and one side; third, on the bottom and two sides, and fourth, on the bottom, two sides and top. Fig. 5 shows the details of the submerged tube 2½ ft. long with the curved approaches on the various sides.

The tubes were supported and braced by pieces of 2 × 4-in. lumber from the bottom, sides and top of the 10-ft. channel, but arranged in such a way as not to interfere with the flow into or out of the tube. The bottom timbers of the bulkhead, holding the upper end of the submerged tube, were 4¾ ins. thick to give the necessary strength, while those nearer the top of the bulkhead were needed only 3¾ ins. thick. In order to have a uniform thickness of 3¾ ins. at the entrance of the tube it was necessary to cut down the thickness of the 4¾-in. planks to 3¾ ins. for a sufficient distance from the entrance to avoid disturbance of the flow. The distance, 7½ ins., shown on Fig. 4, was arbitrarily chosen.

The elliptical approach was selected because of the simplicity in laying out the curve and as it answers the purpose of guiding the water flow as well as would be possible by the use of the more complicated transition curves.

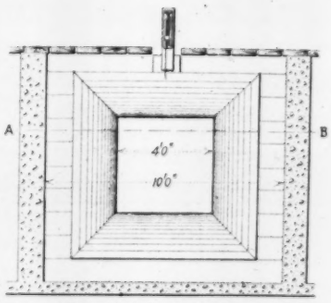
RESULTS OF EXPERIMENTS WITH SUBMERGED TUBES.—The square submerged tubes with square entrance corners and of lengths varying from $^{21}/_{100}$ to 14 ft. have been designated by serial members depending on the length of the tube itself. The subscripts a, b, c and d have been used to designate the particular changed entrance conditions. In this paper the subscript c′ has been used to designate a tube with the same entrance conditions as with "c," but with the outlet varied by the construction of a bulkhead to the sides of the channel as diagrammatically shown for Series 7 c′, Fig. 6. This figure shows in tabulated diagrammatic form the length of tube, forms of entrance used in these experiments, with their proper designations.

The head, h, on the submerged tube has been taken as the difference of level between the surface of water above the bulkhead containing the submerged tube and the surface of the water at the outlet of the tube. The table on p. 37 is a summary giving the coefficients of discharge for the various submerged tubes used in the experiments.

Fig. 6 is a graphical solution of the equations $Q = Ca\sqrt{2gh}$ and $Q = av$, used for submerged tubes and orifices. The ordinates on the left side of the diagram represent values of the cross-sectional areas (a) and vary from 4 to 120 sq. ft. The abscissae marked on the lower part of the diagram represent values of the head (h) on the submerged tube or orifice. These values of (h) have been plotted on horizontal lines, each line representing values corresponding to a certain value of the coefficient of discharge (C), as shown on the lower right-hand part of the diagram. The abscissae marked on the upper horizontal line of the diagram represent values of the average velocities (v) in the cross-sectional area (a) of the orifice or tube. The oblique lines in the upper part of the diagram represent values of the discharge (Q). The diagram is used like an ordinary diagram, having rectangular coordinates, except that the line on which the abscissae (h) is to be read varies in position and depends on the value of (C). Having obtained the coefficients of discharge for certain entrance conditions, lengths of tube or combination of entrance and length conditions, the diagram may be entered with such values and (Q) and (h) determined for these conditions.

The diagram may be used in a variety of ways: Values of (C) and (h) may be assumed, as, C = 0.75 and h = 0.10. The vertical line through h = 0.10 will intersect lines representing Q, any value of which may be selected, and the corresponding value of (a) determined by the horizontal line through the point of intersection of the vertical line and the line representing Q, as, for Q = 50, a = 26.3. By producing the vertical lines to the horizontal line on the upper part of the diagram values of (v) may be determined, thus, for the above example, v = 1.91. The diagram may also be used by assuming values of (a) and Q or (v) and Q and the resultant values of (h) read on the horizontal lines corresponding to some value of (C) as shown on the lower right-hand part of the diagram.

The diagram will be found of special service in preliminary study where it is desired to study the relative effects of changes in a portion of the equation $Q = Ca\sqrt{2gh}$, as the effects on (h) of certain changes in C for a given area of cross-section and discharge.



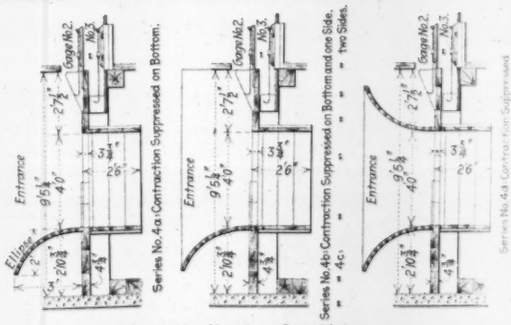

Gage No.2
No.3
Entrance
Ellipse
Series No.4a: Contraction Suppressed on Bottom.
Series No.4b: Contraction Suppressed on Bottom and one Side.
Series No.4c: Contraction Suppressed on Bottom and two Sides.
Series No.4d: Contraction Suppressed on Bottom, two Sides and Top.
Longitudinal Sections.

FIG. 5. DETAILS OF ENTRANCES TO SUBMERGED TUBES, UNIVERSITY OF WISCONSIN EXPERIMENTS.

RECENT HIGH-PRESSURE GAS INSTALLATIONS AT AND NEAR SAN FRANCISCO, CAL.*

By E. C. JONES,† M. Am. Soc. M. E.

The rapid growth of the suburbs of San Francisco, towards the south, presents an alluring field for high-pressure gas distribution.

In the early days of gas lighting on the Coast there was a small coal-gas works at San Mateo, 21 miles from San Francisco, and later on an oil-gas plant was built at Redwood City. After the usefulness of high-pressure systems had been demonstrated, a high-pressure line consisting of 2½-in. extra heavy pipe, with line pipe sockets, was laid from Redwood City to San Mateo, 8.2 miles. On this pipe a pressure of from 10 to 30 lbs. is maintained, depending upon the demand for gas. This line feeds into a high-pressure district in San Mateo, which includes 63,490 ft. of 1 and 2-in. pipe. The consumers in this town are served with gas through high-pressure regulators at the inlet of each meter. This permitted the shutting down and dismantling of the San Mateo gas works.

A high-pressure system was also installed at Redwood City, consisting of 5,410 ft. of steel pipe in small sizes. This is used in conjunction with a low-pressure system at Redwood City fed from the gas works by one governor, reducing the pressure to 4 ins. of water. The work of compressing the oil gas at Redwood City was performed by a steam-driven compressor, with suitable governors for regulating the high pressure of San Mateo and the high and low-pressure mains of Redwood City.

During the early part of 1905, Palo Alto, a city of 4,500 inhabitants, located five miles south of Redwood City, was piped for gas with small, high-pressure mains, using individual governors. This work was done by the Palo Alto Gas Co. This new company purchased its

*Slightly condensed from a paper read before the Fifteenth Meeting of the Pacific Gas Association. The complete paper, with illustrations, appeared in the "American Gas Light Journal" for Sept. 30, 1907.

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supply of gas from the Redwood City plant. To supply this district, 3-in. outside diameter steel tubing, 26,365 ft. in length, was laid in the country road, and four storage tanks, 29½ ft. in length by 4½ ft. in diameter, were installed at the Palo Alto end of the line. This installation was completed May 11, 1905.

In casting about for a suitable place to locate the storage tanks at Palo Alto, it occurred to the writer that it was not necessary to purchase or rent real estate for the purpose of placing gas storage tanks above ground, as many gas companies all over the world are occupying the streets with gas mains of greater diameter than these storage tanks. The tanks were, therefore, placed under ground in a tandem sequence closely surrounded by earth, the regulators occupying spaces between the tanks in cast boxes with covers flush with the surface of the ground. This arrangement is both safe, inoffensive and economical. This high-pressure line is operated under pressure varying from 40 to 60 lbs., and to insure continuous service it became necessary to install an electrically-operated compressor at the Redwood City plant, retaining a gas-engine-driven compressor as a reserve.

To improve the service, and to permit of better regulation, a storage tank was installed at the Redwood City works, and into this tank all the gas used by Redwood City, San Mateo and Palo Alto was pumped. The gas was liberated through separate governors giving to each of the three places the required pressure. At that time the effect of compression on candle power was being widely discussed and was but little understood. All that was known was that the illuminating value of the gas suffered almost directly proportionally to the increase in pressure.

It was evident that the raising of the pressure to 60 lbs. on the Palo Alto line reduced the candle power 4.6 candles. The compression liquid, depending in quantity on thermal conditions and the amount of pressure, was being drawn out of the drips on the line. This liquid consisted of water, naphthaline, and a high grade of benzol. This benzol represented the cream of the candle power of the gas, and consisted of the least stable and most sensitive hydrocarbons which were squeezed out of the gas by compression.

The writer inferred that the compressed gas at 60 lbs. pressure was nearly free from water, and was, therefore, in the best condition to readily and quickly take back into itself the hydrocarbons which had been squeezed out of it; also, that the hydrocarbons would more readily be taken up and again enter into the composition of the gas than the water would be, because the benzol evaporates so much more readily than does water.

The physical changes in the gas due to compression, and to the restoration of the candle power, are in the following sequence: The compressor squeezes out the water and some of the hydrocarbons, and, in order to replace the hydrocarbons, it becomes necessary to keep the gas in intimate contact with the fluid resulting from compression. To do this the writer designed an extremely simple piece of apparatus, and gave it the name of auto-enricher. It is a simple cylinder, preferably of large size, into which all of the compressed gas and the fluid of compression as they come from the compressor enter at one end at the top, and the gas is taken out at the other end at the top, care being taken that the passage of the gas is slow, and that none of the liquid is allowed to escape from the auto-enricher except as it is taken up by the gas. Such a re-enriching tank was placed under ground at the Redwood City plant, with the result that the candle power at San Mateo, Redwood City and Palo Alto was restored to its original condition before compression.

Further experiments (which were subsequently made the subject of a paper by the writer, read before the American Gas Institute) proved that, by raising the compression, certain physical changes took place in the gas, until at 300 lbs. pressure the gas was completely purified from sulphuretted hydrogen and naphthaline, these going down with the compression fluid and not reappearing in the gas when the benzol is taken up by the gas in the re-enriching tank.

In 1905 the largest oil-gas plant in the world was designed and constructed, on new lines, at a point on the bay of San Francisco in San Mateo County, in close proximity to the San Francisco border line, and was named for our president, Martin Station. This plant was built for the purpose of supplying gas to a number of large gas engines for directly generating electricity. Before the completion of these engines the gas-generating plant was finished and started.

To utilize this enormous plant a 12-in. steel tubing was run along the county roads into San Francisco, and connected with the works of the San Francisco Gas & Electric Co., at the Potrero Station. The length of this line is 4½ miles, and a steam-driven compressor was used for transferring the gas. This proved so satisfactory that it became the nucleus of the largest compression plant for manufactured gas in existence. Two power-driven compressors were installed, each having a capacity of 5,500,000 cu. ft. of gas in 24 hours. Following the growth of this plant, the gas engines, three in number, each of 5,333-HP., were completed and successfully started. Between Martin Station and the south is the high ridge of the San Bruno Mts., but west from Martin Station there is a gap through the range, which permits of a practically level outlet into the county roads leading from San Francisco down the peninsula to the south. It required the earthquake, which is not yet effaced from our memory, to bring out the fact that Martin Station is a desirable point from which to supply gas to the cities and towns of the peninsula through high-pressure mains.

The peninsula towns were directly in the path of the earthquake's greatest destruction, but as the high-pressure mains and nearly all the distributing mains in this section were of wrought iron, not the slightest damage was done to the distributing system—this is true of all the high-pressure systems in California. No wrought-iron pipe was broken and no joints were disturbed.

The Redwood City gas works, however, did not fare so well. The boiler settings were badly damaged, and the gas-generating apparatus was moved several inches on its foundation, while all cast-iron connections were more or less damaged. A 20,000-cu. ft. gasholder, in a redwood tank, was completely demolished. This adjustment of matters by the earthquake made it much easier to decide to supply this district from Martin Station.

On Feb. 23, 1907, the work of laying a 4-in. steel tubing from Martin Station to Redwood City, a distance of 125,295 ft., or 23.73 miles, was begun. The inside diameter of this tubing is 3.7 ins. This pipe passes through the following suburban towns, all of which receive gas from it: Burlingame, population 900; San Mateo, 4,600; Belmont, 500; Redwood City, 2,500; Fair Oaks, 250, and the Palo Alto extension, including Menlo Park, with a population of 1,200, and Palo Alto, 4,500, making a total population of 14,450.

This high-pressure line was laid with Dresser couplings, using rubber gaskets, the flanges of the joints being drawn up tightly and evenly to prevent the contact of the gas with the rubber as much as possible. It

VALUE OF THE COEFFICIENT OF DISCHARGE FOR FLOW THROUGH SUBMERGED TUBE, 4 FT. SQUARE, FOR VARIOUS LENGTHS, LOSSES OF HEAD AND FORMS OF ENTRANCE AND OUTLET.

Loss of head, h, in. ft.	Forms of entrance and outlet.	Length of tube, in ft. 0.31	0.62	1.25	2.50	5.00	10.0	14.0
		Values of the coefficient, c						
.05	Square corners.	.631	.650	.672	.769	.807	.824	.838
	a	.672	....	....	.742	.810	....	.848
	b	.740	....	....	.769	.832	....	.862
	c	.834	....	....	.769	.875	....	.890
	c′	....	....	....	....	....	....	.875
	d	.948	....	....	.943	.940	.927	.931
.10	Square corners.	.611	.631	.647	.718	.763	.780	.795
	a	.636	....	....	.698	.771	....	.801
	b	.685	....	....	.718	.791	....	.813
	c	.772	....	....	.718	.828	....	.841
	c′	....	....	....	....	....	....	.830
	d	.932	....	....	.911	.899	.892	.893
.15	Square corners.	.609	.628	.644	.708	.758	.779	.794
	a	.630	....	....	.689	.767	....	.803
	b	.677	....	....	.708	.787	....	.814
	c	.765	....	....	.708	.828	....	.839
	c′	....	....	....	....	....	....	.829
	d	.936	....	....	.910	.899	.893	.894
.20	Square corners.	.609	.630	.647	.711	.768	.794	.809
	a	.632	....	....	.694	.777	....	.819
	b	.678	....	....	.711	.796	....	.833
	c	.771	....	....	.711	.838	....	.856
	c′	....	....	....	....	....	....	.846
	d	.948	....	....	.923	.911	.906	.905
.25	Square corners.	.610	.634	.652	.720	.782	.812	.828
	a	.634	....	....	.705	.790	....	....
	b	.683	....	....	.720	.809	....	....
	c	.779	....	....	.720	.854	....	....
	c′	....	....	....	....	....	....	....
	d	.965	....	....	.938	.928	....	....
.30	Square corners.	.614	.639	.660	.731	.796	.832	.850
	a	.639	....	....	....	....	....	....
	b	.680	....	....	....	....	....	....
	c	.788	....	....	....	....	....	....
	c′	....	....	....	....	....	....	....
	d	.984	....	....	....	....	....	....

Forms of entrance and outlet used for tubes.

Square corners:—Entrance; all corners 90°.
Outlet; tube projecting into water on down stream side of bulkhead.

a:—Entrance; contraction suppressed on bottom.
Outlet; tube projecting into water on down stream side of bulkhead.

b:—Entrance; contraction suppressed on bottom and one side.
Outlet; tube projecting into water on down stream side of bulkhead.

c:—Entrance; contraction suppressed on bottom and two sides.
Outlet; tube projecting into water on down stream side of bulkhead.

c′:—Entrance; contraction suppressed on bottom and two sides.
Outlet; square corners with bulkhead to sides of channel preventing the return current along the sides of the tube.

d:—Entrance; contraction suppressed on bottom, two sides and top.
Outlet; tube projecting into water on down stream side of bulkhead.

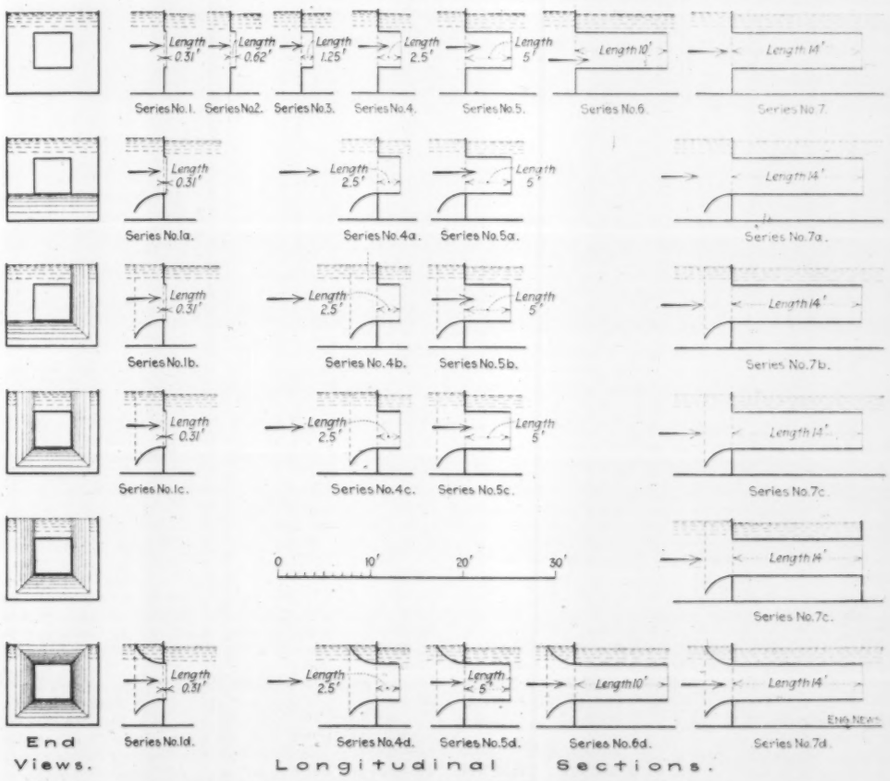

FIG. 6. DIAGRAMMATIC TABLE OF VARIOUS ARRANGEMENTS OF TUBE LENGTHS AND ENTRANCE CONDITIONS.

Series Nos. 1a—7a: Entrance; contraction suppressed on bottom.
Series Nos. 1-7: Entrance; Full contraction.
Series Nos. 1b—7b: Entrance; contraction suppressed on bottom and on one side.
Series Nos. 1c—7c: Entrance; contraction suppressed on bottom and two sides.
Series Nos. 7c¹: Entrance; contraction suppressed on bottom and two sides.
Outlet: Bulkhead to sides of channel.
Series Nos. 1d—7d: Entrance; contraction suppressed on bottom, two sides and top.

is quite important, in using this patent coupling, that the tightening of the bolts shall proceed evenly, so that the flanges may remain parallel. Should one side of the flange be drawn up more than another, the pressure on the rubber gasket is uneven, and the flange is apt to bind on the pipe so that a leak is almost certain.

The painting of this line was a matter of some study, as it had previously been found that red lead and linseed oil of the best quality, although offering a good protection to the pipe, would not withstand the attacks of either the air or the soil. This pipe was, therefore, first thoroughly cleaned and painted with one coat of red lead and oil, then given a second coat of a good and well-proven metallic gas-holder paint which dries with a hard, shiny finish. It is hoped that this covering will be more durable than those formerly used.

A new pattern of drip-box was used on this line, and 25 of them were located at various low points throughout the length of the pipe. High-pressure valves made for the purpose of holding ammonia were placed in the line at five different points, at the approach of each town. These valves were connected into the line by means of an adapter to permit the connecting of a Dresser coupling with a flanged valve. The drip-box is provided with half a Dresser coupling on each side to facilitate its being connected into the line.

No other fittings are used in this line with the exception of six crosses, one at each town. These crosses are provided with adapters to Dresser couplings, and all right angles are turned by bending the pipe at a uniform radius of 20 ft.

At the completion of each day's work the line was capped and air pressure applied to it, up to 100 lbs. per sq. in. This pressure was allowed to remain on the line the entire night. Testing for small leaks was done by the use of soap-suds and a brush, the soap-suds being applied to all parts of the coupling and to the pipe for a distance of 2 ft. each side of the coupling, as this seems to be the vulnerable place where the pipe is apt to be split.

One of the details developed in connection with testing for leaks is the fact that thick soap-suds which has been mixed for several days is better than soap-suds quickly and imperfectly mixed. The importance of careful testing is evident when it is stated that 182 defective sleeves and 29 lengths of split pipe were discarded on this line.

Much of the work was done during uncertain spring weather, and a plow could not always be used to advantage, so that it was necessary to do much of the work with pick and shovel. A grader was used for refilling, and all of the trench after being filled was carefully sprinkled and rolled.

The line was completed June 1, 1907. The actual working days required were 75. The average amount of pipe laid each day was 1,660 ft., while the greatest amount laid in a single day was 3,200 ft. The average depth of the pipe is 2½ ft. The line was placed in commission Thursday, July 18, 1907, and not a single leak developed in any part of the line.

The gas is pumped into the line by a 14 × 16 × 12-in. steam-driven compressor, with a mechanical efficiency of 85%, requiring 55 HP. to drive it, under the present conditions. This compressor has a capacity of about 22,500 ft. per hr. The gas is first forced through an auto-enricher tank, alongside of which is a storage tank.

These tanks are 6 ft. in diameter by 32 ft. long, and have a capacity of 875 cu. ft. The line itself has a capacity of 9,375 cu. ft. The 25 drips on the line have a combined capacity of 19 cu. ft., and the receiving tank at Redwood City, 380 ft. The total capacity of the tanks and line is 11,524 ft. The gas is received at Redwood City into a storage tank 5 ft. in diameter by 26 ft. long. This tank is used for receiving all the gas, and from it the various lines are fed through governors.

Four compression tanks, 6 ft. in diameter by 32 ft. long, are located at Redwood City, as a reserve storage to be used in the event of an accident to the compressing machinery, or on the pipe line. The gas is taken from the receiving tank and pumped by means of a motor-driven compressor into the four storage tanks at a pressure of 80 lbs. per sq. in. This boosting arrangement is a temporary expedient for present convenience, and is capable of many changes, as the storage tanks may be filled directly from the line and discharged automatically through the present governors whenever there is a drop in the pressure. At the present time, approximately 100,000 cu. ft. of gas per 24 hrs. is consumed through the Redwood City high-pressure line, and the line is so free from defects, and the storage capacity of the line and the three tanks (disregarding the four storage tanks at Redwood City) is so large, that the compressor at Martin Station is only operated a portion of the time.

This 4-in. O. D. line has an actual capacity at five atmospheres or 58.8 lbs. pressure of 57,620 cu. ft., or an available capacity at that pressure of 46,096 ft. In speaking of atmospheres in reference to pressure, it must be remembered that when we say that we have five atmospheres in a tank, we do not mean five available volumes of the tank, but four available volumes, and 58.8 lbs. pressure, there always being one volume of atmosphere that is not available or apparent on the gage, and which cannot be gotten out except by exhausting the tank to a vacuum, or filling it with water.

This line completely installed cost 1½ times as much as a 2-in. outside diameter line which would have an actual inside diameter of 1.782 ins. It has an equal holding capacity of 4.3 times as much as a 2-in. D. line, and its carrying capacity at equal pressure is 5.75 times that of the 2-in. O. D. line.

This installation is amply large to provide for the growth of the southern suburbs of San Francisco for many years, and will insure excellent service to the best class of gas consumers. The average monthly consumption per meter of these consumers being 4,500 ft., which is in excess of that of most of our large cities.

In conclusion, the writer may be permitted to apply a very old saying to high-pressure gas distribution—that, like fire, it is a good servant, but a bad master.

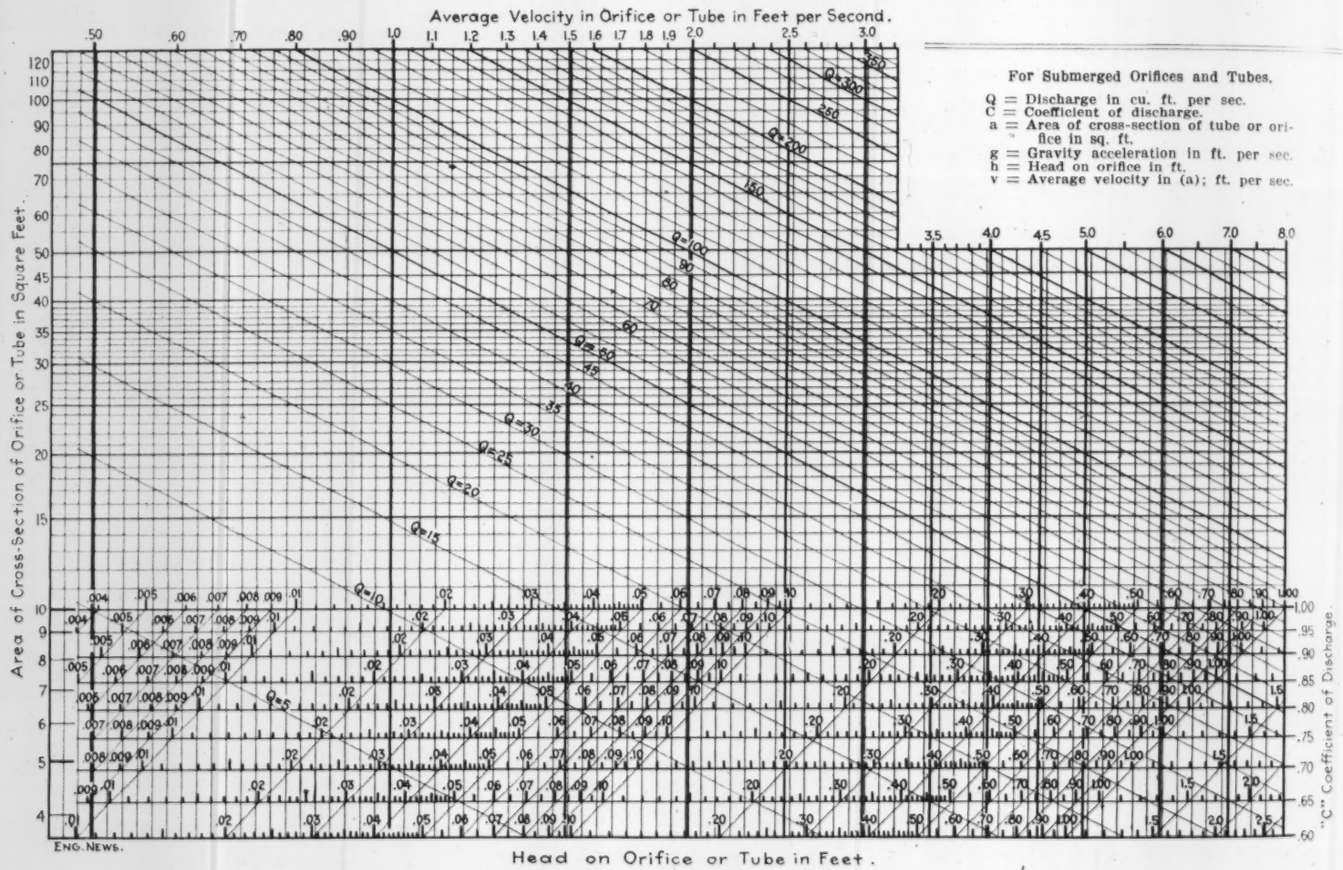

FIG. 7. DIAGRAM FOR THE GRAPHICAL SOLUTION OF THE EQUATIONS $Q = C a \sqrt{2 g h}$ AND $Q = a v$.

PEAT-PAPER MAKING in Sweden has been undertaken on a commercial scale by a concern capitalized at over a million dollars. The money for the new enterprise is stated to come largely from London, but the process is covered by American patents. Extensive purchases have been made and plans have been prepared for mills which will turn out wrapping paper and cardboard. It is claimed that a ton of paper worth $30 can be made at a total cost of $15 in two hours. An account of the launching of the enterprise is given by United States Consul R. S. S. Bergh of Gothenburg, Sweden.

AN INTERNATIONAL CONGRESS OF REFRIGERATING Industries is now being organized under the patronage of the French ministers of agriculture, commerce and industry, colonies and the marine with the idea of holding meetings in Paris in July, 1908. According to a recent report of U. S. Consul-General Mason of Paris the purpose of the congress is to bring together leading experts and representatives of the principal industries and enterprises of all countries in which refrigeration is used as an agent for facilitating the preservation and transportation of food materials by land or sea. Through such a conference of practical and scientific men it is hoped to improve and perfect and on certain lines to standardize the industries of cold storage and transportation by the study and adoption of the most advanced methods and appliances which experience has thus far developed, and to thereby extend the range and promote the efficiency of refrigeration as a factor in international commerce.

An extensive list of papers has been prepared.

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The article on high pressure gas distribution at and near San Francisco, reprinted elsewhere in this issue, is interesting because of the pressures and distances involved, the use of steel mains, and the large compressors and gas engines employed. Incidentally a good word is said for the way wrought iron mains, as compared with cast iron, stood the great earthquake. The small sizes of gas mains now being used under heavy pressures suggest future relief to already overcrowded sub-street spaces. The evident tightness of high pressure mains also suggests the possibility of reducing the enormous and dangerous gas leakage in many American cities from mains under the very low pressures generally prevailing. Certainly if gas can be conveyed without leaks at 50 to 60 lbs. pressure, then mains under a pressure of only a few ounces could be made tight if gas companies only saw or were forced to believe that making them tight is worth while.

After May 1 next railway companies must go out of the coal mining business and out of every other business as well that is not incidental to their business as common carriers. The Federal law reads as follows:

From and after May 1, 1908, it shall be unlawful for any railroad company to transport from any state, territory or the District of Columbia to any other state, territory or the District of Columbia or to any foreign country any article or commodity—other than timber or the manufactured products thereof—manufactured, mined or produced by it or under its authority or which it may own in whole or in part or in which it may have any interest, direct or indirect; except such articles or commodities as may be necessary or intended for its use in the conduct of its business as a common carrier.

The law relates of course to interstate business solely; but several states have similar laws, and Pennsylvania, we believe, has even a constitutional prohibition against railways engaging in the coal mining industry.

That railways engaging in the coal industry are from the nature of the case bound to entirely monopolize it is evident enough; and the attempt to restrict railways to their business as common carriers solely appears a move in the right direction. It has not yet been determined, however, in what way the railways can legally rid themselves of their coal properties, be they ever so well disposed to obey the law. There are no customers ready to purchase outright for cash a good share of the coal mining properties of the United States; and even if there were, these coal properties are largely covered by mortgages running for long terms of years to secure bond issues. If the railways were to obey the letter of the law and merely quit the business of coal mining and transportation, there would instantly be a coal famine. Apparently the railways are relying on this aspect of the case to force either a repeal of the law or such judicial interpretation as shall nullify it. The most practical way out of the difficulty would appear to be for each railway to lease its coal properties to a subsidiary corporation, and distribute the stock in the corporation to present railway stockholders. Such a distribution would not appear in conflict with the law above quoted.

Mr. Sanford E. Thompson's report on reinforced-concrete chimneys, printed in this issue, should be reassuring to those who have been alarmed by reported failures of such chimneys. It places the blame for the failures squarely on poor workmanship and asserts with all confidence that satisfactory chimneys can be built of this material if the precautions required in all good reinforced-concrete work are observed. Incidentally the report makes some recommendations as to principles of design, but it rather seems to dodge the question of what connection there may be between the recorded failures and the design of the chimneys which failed. Perhaps they were all designed correctly; but Mr. Thompson's insistence on the need for proper design suggests that this was not the case.

The matter of heat expansion in chimneys and the stresses resulting therefrom is much more intricate than Mr. Thompson's brief reference to it implies. A chimney is built at ordinary temperatures, and later is subjected to fairly high heat inside while its outer surface remains quite cool. The heated inner surface of a chimney must tend to expand considerably, and this tendency is resisted by the outer part of the chimney, which has a temperature not far above that of the external air. In consequence the inner surface of the chimney suffers a circumferential compression, while the outer surface is under tension. As concrete or brick masonry is strong to resist compression but comparatively weak in tension, cracks may be expected in the outer surface of a masonry chimney if the interior temperature is at all high.

Mr. Thompson says that reinforcing rings of steel should be placed near the outer surface of the chimney to resist these temperature stresses; but he gives no hint as to how these stresses may be computed, so as to correctly proportion the reinforcing rings. In brick chimneys, hoops are built into the brickwork in some cases; but we do not know of any serious attempt to proportion these hoops to the temperature stresses. It may be argued that no greater care than this is needed with concrete chimneys; but in brick chimneys temperature cracks are apt to so distribute themselves at the mortar joints as to be unnoticeable, whereas in concrete chimneys concentration of the cracks is possible.

A few words regarding the general method of computing the external temperature stress in chimneys may make the matter clearer to some of our readers:

Assumption must first be made as to the temperature gradient between the inner and outer surfaces of the chimney wall and as to the coefficient of elasticity of the material. When the chimney becomes heated inside to its normal working temperature, the temperature rise in its wall may fairly be assumed to vary in linear ratio with the distance from the inner face. If the diameter of the shell be very large compared with its thickness, and a straight-line stress-strain law be assumed, the tendency to expansion would result in a neutral line in the middle of the thickness, compression inside of this line and tension outside. Since the chimney increases in circumference by an amount corresponding to the temperature at the neutral line, the extreme-fiber tension and compression would each be due to half the temperature rise from outer to inner face of shell, and numerically would be equal to the product of the modulus of elasticity by the coefficient of expansion multiplied by half the temperature difference between outside and inside.

To illustrate by a numerical example: assume concrete with a modulus of elasticity of 2,000,000, a coefficient of heat expansion of 6 parts per million per degree Fahrenheit and a difference of 400° F. between inner and outer surfaces of the chimney wall. Then the tension stress on the outer surface and the compression stress on the inner surface will be:

$$2{,}000{,}000 \times 0.000006 \times 200 = 2{,}400 \text{ lbs. per sq. in.}$$

The problem is how to reinforce against these temperature stresses. Mr. Thompson does not solve this problem, but says simply:

The stress on the outside surface must be met by increasing the amount of circular steel and placing it near to this outside surface . . . . although not nearer than 1½ ins.

When the total shell thickness is only 4 to 5 ins. (as frequently designed for the upper sections of reinforced-concrete chimneys), the steel cannot be shifted very far from the middle line if this specification be adhered to. But it must be observed that, so long as no tension cracking has taken place, circumferential steel located in the middle line of the shell thickness does not help to resist the expansion stresses. For, the middle being approximately the neutral line of the concrete, the steel at that point also is without stress. It is only after tension cracks have been formed on the outside that the neutral axis shifts inward and the steel becomes effective to take tension and protect against further cracking.

While the above analysis lays no claim to completeness or exactness, it does indicate the importance not only of ample reinforcing against temperature stresses in concrete chimney construction, but of provision to reduce the temperature differences which cause these stresses. The value of interior linings carried clear to the top of the chimney is indicated, with provision for the entry of air to the space between the outer and inner shells.

To ignore the temperature stresses in concrete construction will not be for the permanent advantage of the industry. If it is reasonable to provide for temperature movements in concrete retaining walls, as is now common practice, how much more is it needful where the high temperatures due to artificial heat must be sustained.

SEWAGE SPRINKLERS FOR PERCOLATING FILTERS.

The sewage disposal problem that has been a veritable nightmare in England for fifty years is now almost as troublesome in some parts of the United States and is bound to become more and more absorbing with the increased agitation against the pollution of our streams, lakes and ocean fronts by municipal sewage and trades wastes. Wherever purification is necessary the demand is for higher and still higher rates of treatment, in order to lessen the area required for disposal works and the costs of construction and operation. Practically all systems that do more than clarify sewage include some kind of biological or filter bed to afford an opportunity for myriads of bacterial scavengers to aid in transforming decomposable organic wastes into a stable non-putrefying condition. It is for such beds that high operating rates are demanded.

There are three types of biological sewage beds or filters in use to-day: (1) Intermittent sand filters, which are beds of sand confined by embankments, and provided with carriers to deliver the sewage at the edges of or over the surface of the beds and with underdrains to collect and remove the sewage effluent. These beds are dosed with sewage and are given periods of rest, the volumes, dosing and rest periods varying with the fineness or coarseness of the sand composing the beds, and with the character of the sewage. (2) Contact beds, composed of gravel, comparatively large particles of broken stone, furnace clinker, coke or other material, also confined by embankments, or else by walls. These beds are filled, stand full and are then given an empty or resting period. (3) Percolating

filters, the main characteristics of which are given a few lines further on.

The demand for sewage filters of high filtration rates puts a number of problems before the engineer. Some of the most interesting of these pertain to the distribution of sewage over percolating filters, or that class of very coarse-grained filters upon which sewage is sprinkled in evenly distributed drops, and through which it percolates or trickles in small streams, spread as evenly as possible in thin films over the surface of the lumps of material composing the bed, the liquid meanwhile being exposed to large volumes of free air held in the capacious interstices of the beds. The high rate sought in these filters, 1,000,000 to 2,000,000 gals. and upwards per acre per day, makes an even distribution of the sewage of great importance, since otherwise some portions of the beds may be dosed at an enormous rate, while others receive little or no sewage.

Inasmuch as percolating filters are growing in favor everywhere and have already been adopted widely in this country, their attainable efficiency of distribution is a matter of great importance. Extensive studies of the efficiencies of these types of fixed sprinklers have been made at the Sewage Experiment Station of the Massachusetts Institute of Technology. A condensation of a lengthy paper on the Institute experiments appears elsewhere in this issue. These investigations deserve, as they will doubtless receive, the attention of all concerned with percolating filters.

With intermittent sand filters the distribution of the applied sewage largely cares for itself. The usual aids to distribution are easily and cheaply-constructed channels resting on or in the tops of the beds. The fineness of the filter sand, the heaviness of the dosing (while sewage is being applied), and the felty layer that forms on the surface of the filter all combine in causing the sewage to pond over the whole or a considerable part of the bed. Therefore, while the distribution of the sewage over intermittent filters may be uneven yet it may serve very well, especially in view of the faculty such a filter has of adjusting itself to variations in the relatively low rates to which it is subjected.

In the case of contact beds, which fall between percolating and intermittent filters in daily rates and in size of filtering material and interstitial space, the distribution problem is the simplest of those presented by the three classes of filters. The outlets of contact beds are controlled by gates, and since the beds are filled and held full, the applied sewage can hardly be otherwise than well distributed throughout the whole body of filtering material. The high rate of dosing gives such a volume that the sewage may be distributed over contact beds by means of channels formed in the filtering material itself.

Percolating, trickling or sprinkling filters, as they are variously called, present a far different distribution problem. To begin with, their day-acre-rate is 10 to 20 times that of intermittent sand filters and 2 to 10 times that of contact filters; aeration of the sewage before it reaches the beds is important, even when local conditions do not make it imperative; evenness of distribution is essential on account of the high rates, as already mentioned, and this evenness must be aerial, rather than within the bed, since the coarseness and openness of the filtering material and the velocity of the sewage are all unfavorable to lateral distribution after the sewage strikes the surface of the bed. Unevenness of distribution means loss of utilized area and corresponding rate increase over the area that is utilized. Moreover, should this unevenness be erratic and at the same time intense, the regimen of the beds may be seriously upset by washing out stored matter with the effluent.

At first thought it would seem to be a simple matter to distribute sewage over a filter bed uniformly, but of the great number and variety of distributors thus far devised each has one or more serious drawbacks in the way of unevenness of distribution, liability to partial or complete stoppage, or high cost of installation or of operation. Some of these drawbacks vary with local conditions, and particularly with (1) the freedom of the sewage from clogging material, which depends chiefly on preliminary treatment and therefore is more or less controllable; with (2) lowness of temperature and amount of snow, which are obviously beyond control; and with (3) the available head of the sewage at the point of application. Right here it may be said that in Great Britain, where there are perhaps 50 or even 100 percolating filters to one in any other country, winter conditions are for the most part very favorable, as compared with all the northern portion of the United States, and the available head is far more within control than it is in the United States, since the sewage is quite commonly pumped there and is almost always got rid of by gravity here.

While, for reasons that will soon appear, this article will finally deal with only one class or part of a class of distributors, it may be of service to those who are unfamiliar with the subject if a glance is taken at each of the main classes. Percolating-filter distributors may be divided into two great classes, movable and fixed. The second, or fixed sprinklers, either spray the sewage by means of nozzles or of splash disks served by pipes, or else cause it to fall, rainlike, from multi-perforated carriers suspended above the entire filter bed. The movable circular or radial-arm distributors are generally driven by the force of the escaping sewage, on the Barker-mill or lawn-sprinkler principle, and when this is the case some head besides that necessary to bring the sewage to the discharge level is required. If motor-driven, which thus far has been rarely the case, only a delivery head is required for the circular or radial sprinkler. The rectilinear or rectangular sprinkler must always be power driven. Power-driven sprinklers of either type must be maintained in a level position, and generally require other expensive features of construction or maintenance, besides the expense of providing and operating the motor. The sewage-driven revolving sprinklers have obvious advantages which need not be described or discussed. All the movable distributors, unless of the heavy, awkward and expensive overflow-trough type, are liable to clogging of the holes in the sprinkler arm or pipe. All the movable sprinklers are also liable to stoppages through failure of their motors. The circular distributors involve heavy expense for enclosing walls, since diameters of 100 to 140 or 150 ft. are the maximum. The rectangular movable sprinklers require supports for their ends, which supports may or may not be as expensive as enclosing walls, but must be substantial enough to afford good track alinement and grade.

The sub-class of fixed distributor which covers the whole bed with a rainlike dose utilizes for the purpose a nearly continuous series of specially rolled corrugated iron sheets, perforated with a multiplicity of holes, through half of which nails protrude to aid in the distribution. The units of this system must be set and maintained at a level. The system might be adapted to beds of any shape or size, but of course it is best suited to rectangular beds and thus far it has been applied to relatively small areas, only.*

The other sub-class of fixed distributors, including spray nozzles of many types and also spray disks, may be adapted to beds of any shape and size. Each one of these sprinklers wets a circular area, and any group of them leaves a portion of the gross area served by the group either (1) unwetted, or (2) overwetted, or (3) underwetted—and this without regard to the distribution of the sewage over the individual wetted areas. We shall come back to this matter shortly. To utilize either the nozzle or the disk sprinklers a head sufficient to deliver the sewage a few feet above the surface of the filter bed is required. The nozzle sprinklers deliver the sewage upwards, fountain-like. The disk sprinklers deliver it downwards onto the disks, from which it splashes in every direction, also somewhat fountain-like in effect. Either the nozzles or the disks require a network of feed pipes to supply them. The greater the area wetted by each sprinkler the fewer the sprinklers and the shorter the total length of feed pipes will be, but with variations in individual wetted areas there are variations in efficiency of sewage distribution.

At the present time movable sprinklers of the revolving or radial arm sub-class are more extensively used in Great Britain than any other kind of percolating-filter distributor. Next come the fixed nozzle sprinklers, at least if size of installation be considered. We are not certain whether the rectilinear movable or the overall-area fixed sprinkler ranks next in amount of use. The fixed disks, so far as we know, have never been used abroad, and neither they nor any of the other kinds of sprinklers under discussion have yet been used in the United States, so far as we know, except in experimental plants. American experiments, if we are correctly informed, have been confined wholly to revolving and to either nozzle or disk fixed sprinklers, and chiefly to the two sub-classes just named. Revolving sprinklers are now being tested at Baltimore. Fixed-nozzle sprinklers were tested at the sewage experiment station at Columbus, O., and are to be used in the large city plant now under construction there.

For the sake of completeness, perhaps we ought to state that tipping trays have been tested for some years in Massachusetts, both at the Lawrence and the Institute of Technology experiment stations. We could never understand why this was done and are pleased to note that they have been abandoned at the Institute.

Fixed sprinklers of the nozzle or disk type, it appears, are most likely of any of the classes to come into early and extensive use in the United States. They seem better adapted to American climatic conditions than do any of the other classes of sprinklers, and they seem likely to prove more economical for the large filtration areas required to treat the large volumes of sewage flowing from American cities. Moreover, American engineers appear to think that the movable sprinklers would prove undesirable through stoppages.

The efficiency of nozzle and disk sprinklers depends on not only (1) the evenness of distribution of sewage within the circular area wetted by each sprinkler but also upon (2) the ratio of (a) wetted or (b) overwetted area to total area; (a) depending upon how much space is left between the wetted circles, while (b) depends upon the overlapping of the circles. In the studies carried on by the Institute a "crude coefficient" was used to express efficiency of one sprinkler of a given type and a "corrected coefficient" to represent the efficiency of a group of sprinklers. The crude coefficient of the six types of sprinklers studied ranged from .44 to .80 and the corrected coefficients from .22 to .80. One sprinkler showed the same crude and corrected coefficient, of .80, and one gave a range of from .73, crude, to .22, corrected. These conclusions, if they are verified by those of other experiments and by the results attained in practice, forcibly illustrate the importance of studying distributor efficiencies.

We think it open to question whether the comparison by corrected coefficients is perfectly fair, since it was based on a uniform day-acre-rate of 2,000,000 gals., thus putting at a disadvantage sprinklers designed for dosing at a higher rate, since, to give the lower rate, the sprinklers are spaced farther apart than was intended when they were designed, and therefore a large increase in unwetted area occurs. In practice, of course, an intelligent engineer would increase his number of sprinklers and at the same time decrease his rate of discharge per sprinkler, if he wished to cut his dosing rate in half. That is, he would do this if he had intelligently used such facts as are brought out by these studies. It must be borne in mind, however, that changes in nozzle-discharge capacities can only be brought about by changes in design or

*This system is known as the Stoddart, after F. Wallis Stoddart, County Analyst, Bristol, England.

operation that will more or less notably effect the efficiency of a given nozzle. This suggests the desirability of carrying the Institute tests further, adapting each sprinkler to a uniform dosing rate, then finding its optimum head, and at last determining its efficiency.

It should be noted that the studies of the Institute, like most other work at sewage experiment stations, does not take into account the relation of the various sprinklers and their efficiencies to the cost of building filter beds. If the required number of sprinklers per acre, given in the table near the close of our reprint, be taken into account the cost of beds equipped with the various sprinklers might be materially affected thereby. For instance, 667 sprinklers (on the face of the table) are required for a type giving .67 efficiency and only 341 for a type giving .62 efficiency, besides which the design of the two types is radically different. It must be remembered that an increase in number of sprinklers is only a beginning in the increased cost of a filter bed, since the amount of piping must also be increased.

The ultimate effect of imperfect distribution of sewage on percolating filters is either a decrease in the purification effected or else an increase in the required area. Sometimes both effects result. The experiments at the Institute station were not designed to determine these factors. Their object was to test the best forms of and working conditions for a type of sprinkler peculiar, so far as we know, to the Institute, and then to compare the results with those obtained from other types. This sprinkler consists of a disk upon which a stream of sewage is discharged vertically and from which the sewage is discharged in the form of a spray. Disks having a great variety of shapes were tested in order to determine the best form. Tests were also made to learn the optimum height from which the sewage should be discharged onto the disk and also the most suitable distance between the disk and the surface of the bed. The best of these disks gave high crude and corrected efficiencies, and would require a number per acre about half that for the most efficient nozzle type and nearly four times the number for the least efficient nozzle (on the face of the tabulation). No figures bearing on the relative costs of installating and operating disk and nozzle sprinklers are given. A single disk would be much less expensive to make than a single nozzle, but the installation of the disks involves expense not necessary to the nozzles, since the disks *must* be supported above the beds and must be fed from pipes at a still higher elevation, while the nozzles *may* be screwed into pipes resting on the surface of the beds. In the matter of operation, however, the disks might have the advantage of clogging less than even the American sprinklers, with their very free and capacious waterways. In winter, it might be a toss-up between the troubles from ice with the disk and with the nozzles, each type having advantages peculiar to itself. It seems safe to say that during severely cold weather anything gained by requirements in the shape of the disks would be negatived by ice formation; but the winter operation of any kind of filter-bed apparatus is likely to be more or less upset by ice and snow.

Coming back to the general subject of sewage distributors, we shall look forward with great interest to the results obtained at Columbus when the fixed nozzles being installed there are subjected to the vicissitudes of every-day work. As to the Institute work, we should like to see tests of the efficiency of distribution of some of the movable distributors and also of the Stoddart fixed sprinkler, with its simultaneous distribution over practically the entire filter area. In addition, we should like to see the results of tests of disk and nozzle sprinklers during high winds and extremely low temperatures.

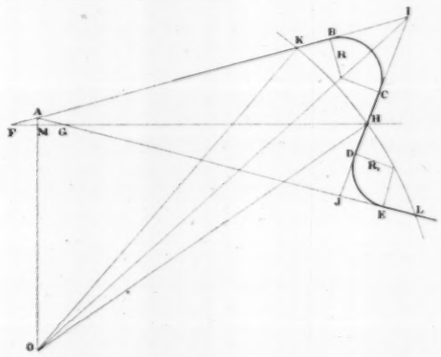
Finally, the Institute is to be congratulated for breaking away from the traditions that have hampered the otherwise excellent work of the Lawrence Experiment Station and making comparative studies of mechanical devices in actual use.

LETTERS TO THE EDITOR.

A Railway Location Problem.

Sir: The following problem in railway curves has generally been evaded owing to the difficulty in obtaining results by direct methods. The solution below is reasonably simple.

PROBLEM: Given two intersecting tangents AB and AE, of known lengths, and the intersection angle BAE. Determine the two equal radii R and R, of the curves of the crossover BCDE such that the intervening tangent CD shall have a given value.



SOLUTION: Make $AF = AG = \frac{1}{2}(AE - AB)$; FG will bisect CD at H. Make $KB = CH = HD = EL$. Draw AO perpendicular to FG. Let O be the center of the circular arc passing through both K and L; this arc will bisect CD at H. $\angle AFG = \frac{1}{2} \angle BAE$.

$OA = (AG + CH) \operatorname{cosec} \angle AFG$. Find AM and OM.

In triangle OMH, find $\angle MOH$. $\angle KOH = \angle MOH - \angle AOK$. $OH = OK$.

In triangle OMH, find $\angle MOH$. $\angle KOH = \angle MOH - \angle AOK$. $\angle KOI = \frac{1}{2} \angle KOH$. Find $\angle KIO$.

In triangle OKI, find KI. $BI = CI = KI - CH$.

$R = R_1 = BI \tan \angle KIO$.

CHECK: $\angle KIJ = 2 \angle KIO$. In triangle AIJ, find $\angle AJI$, IJ and AJ, thus obtaining DJ and JE, which should be equal. Solve for R_1. It should equal R.

Yours truly,

C. W. L. Filkins.

770 S. Broad St., Phila., Dec. 10, 1907.

Original and Acquired Roughness of a 30-in. Cement Water Supply Conduit, Southern California.

Sir: A letter in Engineering News of Sept. 26, 1907, entitled "A Courageous Crawler," prompts me in sending the enclosed view in the belief that it will be of interest.

The 30-in. pipe shown is in Southern California, and was placed about 1890. Data as to details of its construction are not available, nor have I at hand notes as to its gradient or original capacity.

Note the normal water line, and below it, on the sides and bottom, the foul condition. This fouling is a hard deposit of "caleche" or lime, common in the arid regions.

Interior View of a 30-in. Cement Water Supply Conduit Showing Original and Acquired Roughness.

(The pipe was laid about 1890, some 20 miles northwest of San Bernardino, Cal. It is about 1.8 miles in length. The view shows the conduit at a point some 800 ft. from the entrance.)

Note also the rough appearance of the deposit—the bottom being almost as bad as the sides; there being no apparent tendency to a uniformity of increment, though a tendency to uniformity of slope is quite apparent, in this case closely approximating 1 to 1.

The primary cause for this condition is no doubt the failure to remove the mortar squeeze at the joints and particles of gravel during the progress of construction; and later to inattention at the intake, permitting a quantity of sand and gravel to enter the pipe at flood times, thus forming countless irregular obstructions. And since this form of deposit is hard and cement-like, the obstructions grow in size, and in time reduce the capacity of the pipe to a great extent; differing in this respect to silt deposits in the irrigation canals of the southwest, for I have seen such channels improved by silting.

The foregoing is preliminary to the query as to the nicety, or degree of smoothness, of the inner side of cement pipes, that should be required. The writer would appreciate the opinion of others who may be interested.

In conclusion, if any should ask how to learn the size of a pipe, "crawl." He may not get the desired information, but he will know comparatively his own girth, and have a soreness quite as interesting.

Yours truly,

W. E. Condon.

1826 Parker St., Berkeley, Cal., Dec. 5, 1907.

Sampling Concrete with Core Drills to Determine Its Quality.

Sir: In a great many instances where contracts have been completed on concrete work, the question has arisen whether the contractor has fulfilled the specifications in regard to mixture, etc., and the result is usually a long drawn out litigation. To determine whether or not the specifications have been followed, is quite difficult if samples are taken by digging into the work, for if the samples are to be of any value it is necessary to take them from various sections and from the interior, even more than near the surface. If the work is a heavy arch, wall or foundation, to dig a large hole into it would not only deface it, but injure its strength.

A Concrete Sample Taken with a Core Drill.

The most practical method for examining the quality of finished concrete work is by sampling the material by taking out a core with what is known as a shot drill. These drills are made up in various sizes, but the small hand power outfits answer the purpose in practically every instance. A hole can be run into the concrete very rapidly, a core removed, analyzed, and the character of the mixture accurately determined; the holes can be easily filled and do not injure the strength or appearance of the work. Samples can be taken at any depth desired.

The writer does not know whether this method of testing concrete was original with the Government, but the first work of the kind he ever saw was on the locks and dams on the Ohio River a few years ago. The accompanying photograph shows a section of a 1¾-in. core taken from the concrete on this Government work.

This method of testing concrete aside from being an absolute check on the contractor, also keeps a check on the inspector, who is placed in charge of the work, or saves the cost of an inspector altogether, as no contractor will willfully deviate from his specifications if he knows his work will be sampled and tested. It also gives the contractor an opportunity to prove beyond question that he has followed the specification to the letter and leaves no room for doubt relative to the character of his work.

B. G. Cope.

Orrville, Ohio, Dec. 14, 1907.

[In this issue we publish an article by Mr. R. L. Wales on "The Analysis of Concrete," which describes a method of analyzing the concrete after the test piece has been taken.—Ed.]

Dust Explosions in Coal Mines.

Sir: The article on "Dust Explosions in Coal Mines" by Mr. J. L. Dixon, in your issue of Jan. 2, 1908, contains sound advice that could, with advantage, be seriously considered by the mine operators of this country. I desire, however, to call attention to one feature of the remedy suggested which to me appears deficient. I quote from Mr. Dixon's article:

> The remedy is simply this: After a room or entry has been undermined by machine and before a shot has been fired, the dust should be thoroughly soaked with water, and the roof, sides and floor of the room should be sprayed for 25 or 30 ft. back from the face.

I doubt whether this would settle the dust which is in suspension and which has already traveled much more than 25 or 30 ft. from the face during the progress of the under-cutting. This suspended dust is the objectionable feature to be eliminated. A more effective way would be to apply a spray or jet of water at the face of the cutting tool while the coal is being cut, as in this way no dust could escape either in the air or scraped on to the floor, as all material broken down would be thoroughly water-soaked. A very simple arrangement for delivering water to the cutting face could be devised, which would absolutely prevent the formation of a dangerous mixture of dust and air. Spraying the working face after the cutting has been done would, of course, lend additional security, but it is far better to prevent the escape of dust than to attempt to render it harmless after it has escaped.

A constant jet of water playing on the fine material would in addition aid wonderfully the health of the miner. A visit to the face of an entry or a room where cutting is in progress will convince the average individual that common sense alone would suggest to the cutter and scraper the adoption of some scheme to lay the dust, not alone for the sake of their own bodily welfare, but also the added security assured to their fellow workmen.

I believe the danger from the use of electricity in mines is much exaggerated. Dust in order to become dangerous must be in an extremely finely divided state and of such quantity that communication between the particles would be intimate. The time of duration of an electric spark is so short that but few particles of dust could be ignited, and it is doubtful whether the heat generated would be sufficient to cause combustion of other particles than those coming directly in contact with the arc. I am of the opinion that an explosive mixture, consisting of coal dust and air, requires a large flame with considerable concussion to start the explosion, such as, for instance, is generated by a blown out shot or by a previous explosion of fire damp. In any case Mr. Dixon's suggestions are timely, and the mining profession must surely have realized that every possible precaution should be adopted, if for no other reason than that of practical economy.

Very truly yours,
G. P. Bartholomew.

Wilmington, Del., Jan. 3, 1908.

A Boiler Explosion in a California Oil Field.

Sir: I enclose photographs of a remarkable boiler explosion which occurred on the Calloma Oil Co.'s lease in the Kern River Oil Field, Kern Co., Cal., on the morning of Saturday, Oct. 19, 1907.

The boiler in question was the north member of a battery of three 70-HP. cylindrical return-flue boilers, weighing about five tons each. The boilers were set in substantial brick settings and were fired with oil. The north member was provided with an independent steam dome, connected to the boiler by a cast-iron flange. The boiler, though not new, was in first-class condition, and free from any appreciable amount of scale on either shell or flues, and was entirely tight, in fact it had been recently overhauled. The pumper (as the man in charge of the boiler is locally called) states that a few minutes before the explosion he had "a gage and a half of water" equivalent to about 5 ins. of water above the upper flues, although it would seem there must have been a serious error somewhere in this statement.

There was no one in the boiler-house at the moment of the explosion, and the immediate cause has not been made clear.

The explosion completely wrecked the north end of the boiler-house, and hurled the boiler itself a measured distance of 1,220 ft., across two gulches, and the intervening spur, clearing the top of a 74-ft. derrick which stood almost in the line of flight, and burying itself between 2 and 3 ft. in the hillside on which it struck. The steam dome flange of cast iron was ruptured, and the dome itself dropped among the debris immediately below its original position, uninjured.

The boiler itself, according to an eye witness of its flight, was red hot, and it certainly was hot enough to fire the dry grass where it struck. Those who arrived on the scene a few minutes later are certain that there was no sign of water, either in the boiler or on the ground.

The shell of the boiler is badly battered out of shape but is otherwise intact, no seams have opened, and a hammer test shows all the rivets to be still fast, the flue heads are torn between holes, and practically all the flues are out, but in the opinion of all who have examined the boiler this damage was done by the shock of the fall as the appearance of the fracture indicated a tearing rather than an explosive action. Providentially no one was injured, although two men were in a bunk-house close by which was wrecked by the flying bricks.

The other two boilers in the battery were uninjured and not disturbed in their connections, although the brick settings were so shaken that it was considered wise to rebuild them.

I would greatly appreciate your opinion and that of some of your readers as to the source of the tremendous lifting power of the explosion, seeing that beyond much question the point of failure was the cast-iron flange on top of the boiler.

Yours very truly,
Ernest H. Wilcox.

728 Bryson Block, Los Angeles, Cal., Nov. 29, 1907.

[Nothing can be more certain than that the boiler did not make its rocket-like flight through the air without adequate propelling force, and the propelling force, in any boiler explosion, is furnished by the reaction of the escaping steam and water, exactly as in the case of a rocket. Unless there is some rent in the bottom of the boiler which escaped attention on account of the boiler being buried in the ground, the explosion must be assigned to failure of the tube plate, due probably to overheating from low water. As soon as these rents occurred in the tube plate, by tearing between the tubes and the tubes themselves pulling through, the steam and water would escape with enormous velocity and would furnish force sufficient to tear the boiler from its setting and start it on its aerial flight. The steam dome with its cast-iron flange would easily break loose from the boiler and would be held by the pipe connections, which explains its present location. Not much importance is to be attached to the statement of an eye-witness that the boiler was red-hot when it passed him. The man was not in a frame of mind for judicial observation. The water in the boiler was probably largely forced out when the rent occurred. The overheated tube sheet which failed might have been hot enough to fire the grass.

It is common enough for vertical boilers to go up like a rocket when the lower tube sheet gives way; but it is seldom that a horizontal tubular boiler takes such a long flight.—Ed.]

FLIGHT OF AN EXPLODED BOILER IN A CALIFORNIA OIL FIELD.

Comparing City Pumping Station's Cost with Private Power Station's Cost.

Sir: I am pleased to note Mr. Irving H. Reynolds' criticism of my comparison of cost of steam power by items in private and municipal works, in your issue for Dec. 5, 1907. The use of the table was incidental to a discussion of municipal ownership and management of public utilities, and was intended to show, as it does show, that the cost of labor per annual horse-power is much larger in municipal pumping works than it is in private power-houses. Mr. Reynolds has kindly given one reason, not political, why it is so.

The objection to comparing public and private pumping power costs, is that the latter, at least in this country, rarely represents any considerable power, either in the units or aggregation, and while the publicly-owned and operated stations are generally economically administered, the objection which Mr. Reynolds points out to the comparison of 1,000-HP. pumping engines to 5,000-HP. electric power engines will be just as strong against the comparison of power costs in a small private works, with a large municipal works; that is, it will take at least one man on watch, at any time, to attend to a small pumping engine developing 100 HP., and the same man can with no large increase of labor attend to an engine in a municipal station developing 500 to 1,000 HP.

Excepting Denver, Omaha, Indianapolis and San Francisco, there are no large cities in this country having privately-owned water-works. Two of those are primarily gravity works, with a little secondary, or high level pumping, and the other two are scarcely of size to be compared to Boston, St. Louis, Cleveland and Chicago.

Mr. Reynolds will kindly observe that I took into account only cities having large pumping units and modern machinery, some of which, I am proud to say, were designed by my critic. Had the comparison extended to smaller cities and ancient pumping engines, the labor charge per annual horse-power for the municipally-owned works would have been appalling.

Mr. Reynolds' argument, in the main, is correct: that the horse-power cost for a power unit of 1,000 I. HP. should be greater than for a power unit of 4,000 or 5,000 I. HP.

Concerning the item of repairs, it seems to me it should be lowest for engines of low piston speed and r. p. m., and I believe experience would so demonstrate if the municipally-owned and operated pumping engines used in the comparison were placed under private control.

However, the table of power cost was only one of several facts cited in opposition to municipal ownership of public utilities.

The reduced cost of labor for annual horse-power in private works is not due to the larger development of power per engine, but must be credited in part, at least, to the different conditions surrounding the employment of labor for public and private works. In private works, the politics or political influence of a candidate for appointment to responsible or ordinary duties, has no weight whatever with the appointing power. No private corporation is running its business for the benefit of any political party, while the candidates, politics, and political influence in his town, ward or voting division, is the "sine qua non" for appointment to a municipal job.

Respectfully, John W. Hill.

Cincinnati, Ohio, Dec. 17, 1907.

The Hydrographic Investigations of the U. S. Geological Survey.

Sir: The U. S. Geological Survey, in connection with its other activities, has been engaged for the past twelve years in making investigations of the quantity, quality and availability of surface and ground water throughout the United States. The results of these investigations have been published regularly as Water Supply and Irrigation Papers and have been available for distribution and for reference in libraries throughout the country. The importance of this work and the value to the public of a knowledge of our water resources has led some of the states, private individuals and corporations to cooperate with the Survey in securing the desired information and the results of many independent measurements, and investigations have been placed at the disposal of the Survey and made available for general use by their publication with the results of the investigations made by the Survey in the Water Supply and

irrigation papers. The value of this information is by no means limited to the communities in which actual observations are made, as the intelligent study of water yield in various parts of the country under known conditions of precipitation, temperature, altitude, topography and cultivation affords a basis for at least approximate determination of the water resources of other localities where no direct investigations have been made, and as the hydrographic investigations are continued throughout the country under the uniform methods and skilled direction of the Geological Survey it may ultimately become possible, with the accumulated data covering annual and cyclic variations in stream flow, to formulate general laws which will permit a determination of the water yield in localities where the water resources have not been investigated, from the meteorological records of the U. S. Weather Bureau and a knowledge of the topography, extent and nature of cultivation and other factors which are readily ascertained.

It hardly seems necessary to draw attention to the absolute impossibility of developing any part of the country without an adequate supply of water of suitable quality. For domestic purposes, for irrigation, for every class of industrial enterprise, a determination of the availability of an ample water supply is the first consideration. We have never heard any question raised concerning the importance of these hydrographic investigations, their value to the people, nor the thoroughness with which the Survey has done its work, within the limitations of the funds available.

These investigations have been continued during the past twelve years, and the appropriations therefor were gradually increased by Congress to $200,000 per annum, which amount was available for four years prior to June 30, 1906. The Committee on Appropriations of the House (59th Congress, 2d session) disapproved of this work and claimed that it had no Federal purpose and should not therefore be supported by the Federal Government. The Chairman of the committee stated that "I do not take issue with the advocates of this appropriation on the question of benefit to the people. The issue which I raise with them is the question of whose duty it is to perform the work." This committee, in reporting the Sundry Civil Bill, made no provision for this work for the year ending June 30, 1907. An attempt was made on the floor of the House to introduce an amendment, the result of which would have been to provide a suitable appropriation, but a point of order was raised and sustained.

The Senate later approved an appropriation of $150,000, which, in conference between the committees of both branches, was reduced to $100,000, and the Survey has therefore been obliged to carry on the work during the present year for half the amount previously available. The amount of work performed has been proportionately reduced.

In regard to the point of order sustained in the House, it was alleged that the organic act of the Geological Survey does not make any provision for these investigations, and that therefore any appropriation for same is without authority in law, notwithstanding the fact that the work has been continued over a period of twelve years under appropriations of Congress.

The Secretary of the Interior has included in his estimate for the year ending June 30, 1909, the sum of $200,000 for hydrographic investigations of the U. S. Geological Survey. The attitude of the members of the present Committee on Appropriations on this subject is not known to us, but in view of the precedent established on the point of order at the last session it is manifestly desirable that a bill be passed giving the Survey specific authority to continue these hydrographic investigations, before the appropriation bill comes up for action.

Such a bill (House Bill No. 6122) was introduced on December 9 by Mr. Needham "to provide for continuation of investigations of the rivers and water resources of the United States" and will presumably be passed upon by the Rivers and Harbors Committee early in January and soon after acted upon by the House. If this bill is passed there will be no possible legal objection to the appropriation for hydrographic investigations in the Sundry Civil Bill, and the item may then be considered by Congress solely upon its merits when this Appropriation Bill is taken up later in the session.

In considering the subject a year ago we wrote as follows:

> It is manifestly impossible for any individual investigators or corporations to pursue such an extended series of gagings and studies of the laws governing the water resources of the United States as is necessary to establish the relations between rainfall, evaporation, temperature ranges and other climatic conditions affecting the water yield of drainage basins in different parts of the country, and where such local investigations are made by private enterprise the information secured does not ordinarily become available to the interested public, except, indeed, when the data is communicated to the Geological Survey and combined with the results of their own work in the publication of Water Supply and Irrigation Papers.
>
> It is equally impracticable for the several States to thoroughly cover these investigations, as there would be such lack of uniformity in methods as to prevent the general application of the data thus independently collected and in the case of the larger streams which bound or traverse several States the net results would probably be so confusing as to be unsafe for practical use.
>
> The increased development of all the natural resources of the country is, of course, desirable, but in the case of water, which is indispensable to the prosperity and even the existence of every community, the most complete knowledge should be available in order that it may be intelligently and effectively conserved and utilized for power, for irrigation and for municipal water supplies.
>
> In our judgment this is properly a Federal undertaking, equally with the maintenance of the Weather Bureau, the control of navigable rivers, the institution of forest reserves, the surveying and mapping of the United States and other public works which cannot effectively be undertaken by individuals or the several States.
>
> We trust that Congress will appreciate the real value and propriety of this Federal undertaking and the seriousness of even a temporary interruption of the consecutive observations which are essential to the proper conduct of the work, and that House Bill No. 6122 may be favorably reported by the Rivers and Harbors Committee, become a law and that the requested item of $200,000 in the Sundry Civil Bill may be favorably reported, and this appropriation thus again become available.

Yours very truly,

Sellers & Rippey.

Stephen Girard Building, Philadelphia, Pa., Dec. 23, 1907.

[This general subject was discussed at length in the editorial columns of our issue of Nov. 29, 1906, and in our issue of Dec. 6, 1906, we published a map showing the location of the 800 gaging stations of the U. S. Geological Survey. In the first of the issues named we deplored the curtailment of the gagings appropriation and urged our readers to lose no time in impressing upon their Congressmen the importance of these gagings to the engineering, industrial and general interests and welfare of the country. Now that the water resources of the country are to be studied exhaustively by an Inland Waterways Commission, the need for systematic, numerous, well-distributed stream gagings is more apparent than ever before.—Ed.]

The Elastic Theory and a Faulty Arch.

Sir: In your issue of Aug. 1, 1907, Mr. R. C. Barnett discussed the application of the elastic theory to reinforced-concrete arches. Mr. Barnett is quite right in his criticism of the usual application of the elastic theory to the masonry arch without including the abutments as elastic, and his suggestions would doubtless improve ordinary practice in applying the elastic theory. I do not believe, however, that any method of applying this theory to masonry arches can be trusted to produce a design at once safe and efficient. There is no question in my mind that the elastic theory is correct; the entire trouble is with the assumptions on which it is based and the difficulty of application.

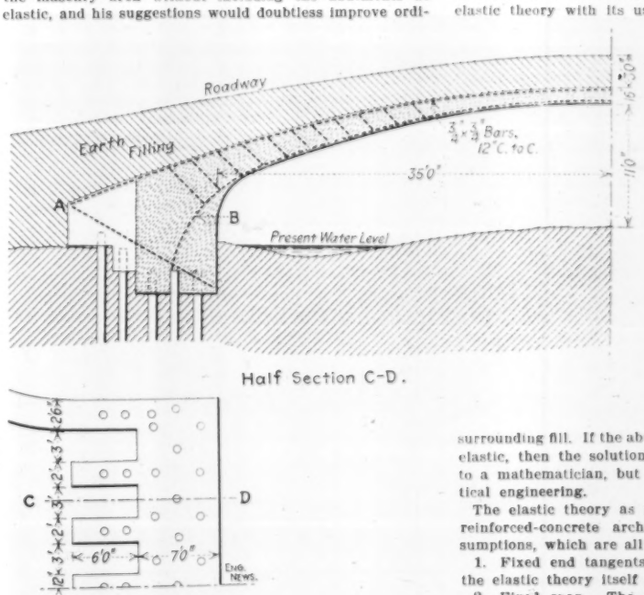


FIG. 1. LONGITUDINAL SECTION AND PART PLAN OF A NEW REINFORCED-CONCRETE PARK BRIDGE.

One of the assumptions is that the end tangents are fixed in direction. Assuming a load to be applied at the crown of a linear arch, a solution by the elastic theory results in maximum moments at the ends of the arch ring. This maximum moment and its resulting stress produces distortion of the arch ring which is transmitted to the supports, and immediately the end tangents shift in direction and our basic assumption is upset. In other words, although we have assumed the end tangents fixed in order to apply the elastic theory, the elastic theory at once proves that the end tangents cannot be fixed. This same condition will apply regardless of whether the arch is supposed to extend from springing to springing or from foundation to foundation.

If the elastic theory produced satisfactory results, the above criticism might be deemed hair-splitting; but, while I am unable to point the particular discrepancy that is important, I am satisfied that somewhere in the wood-pile of the elastic theory, a colored gentleman is concealed. For example, see the latest text on this subject, Prof. Howe's "Symmetrical Masonry Arches," 1906, in which is proved by the elastic theory that a change of temperature of 5° produces as serious stresses in a 60-ft. granite arch as the railway loading; and that a temperature change of less than 40° will wreck the structure. And Prof. Howe, instead of attributing these remarkable results to any defect in the theory or its assumptions, draws the following conclusion:

> As a large number of railway bridges have been built upon practically the dimensions we assumed and no indications of failure have been found, we must conclude that the range of temperature change assumed in this example is very much too great.

This reads very much as though the writer had become imbued with his theory to the obliteration of practical common-sense.

Mr. Barnett asks for measurements taken of deflection of end tangents. It is not at all probable that any one has ever made such measurements for the reason that no one except the most extreme theorist would find them useful. To show that it does occur, however, one need only inspect a concrete arch of the usual earth-covered type having continuous spandrel walls at each side of the roadway. If no expansion joints are provided in the walls from the copings down to the springings, the wall will crack at these points regardless of temperatures; or, if expansion joints are provided they will open when the centers are struck. This ought to be conclusive evidence to any one that the fibers at the inner surface near the springing are in compression while those at the outer surface are in tension, just the reverse of the results obtained by the elastic theory. Nor can any application of a live load to such an arch produce any different results. For an earth-covered arch therefore, there is no occasion for any reinforcement near the inner surface at the springings. And for similar reasons no reinforcement is desirable near the outer surface over the crown. The elastic theory with its usual assumptions would require a reinforcement near each surface of the arch continuously, yet I have personally supervised the design and erection of upwards of 800 reinforced-concrete arches, including spans of 105 ft., in which a single row of reinforcing rods passing near the inner surface at the crown and near the outer surface at the haunches has proved sufficient.

Mr. Barnett's suggested solution for the end tangents by resorting to the fundamental equations is impracticable for the reason that the shifting of the tangents is due to the yielding of the abutments, and the abutments include not only the mass of concrete, but a large amount of surrounding fill. If the abutments were absolutely fixed, yet elastic, then the solution suggested might be of interest to a mathematician, but altogether too tedious for practical engineering.

The elastic theory as usually applied to earth-covered reinforced-concrete arches, involves the following assumptions, which are all more or less erroneous:

1. Fixed end tangents. If the abutments are elastic, the elastic theory itself proves this assumption unsound.
2. Fixed span. The span changes when the centers are struck if abutments are elastic.
3. No tensile stresses in arch. Yet an arch without tension requires no reinforcement, hence the elastic theory applied to a reinforced arch is to that extent inconsistent.
4. Neutral axis coincident with arch axis. They vary materially.
5. Axial stress neglected. Yet in flat arches its omission may decrease the estimated stresses 50%; and the axial stress affects the end tangents.

Add to all the above the fact that for economy of labor in analysis it is almost essential that the loading of earth on the arch be assumed to act vertically, with resulting errors far in excess of any of the above assumptions, and it will be seen that the elastic theory applied to such arches is not a desirable method except in the hands of the most expert.

An interesting example of a reinforced-concrete arch

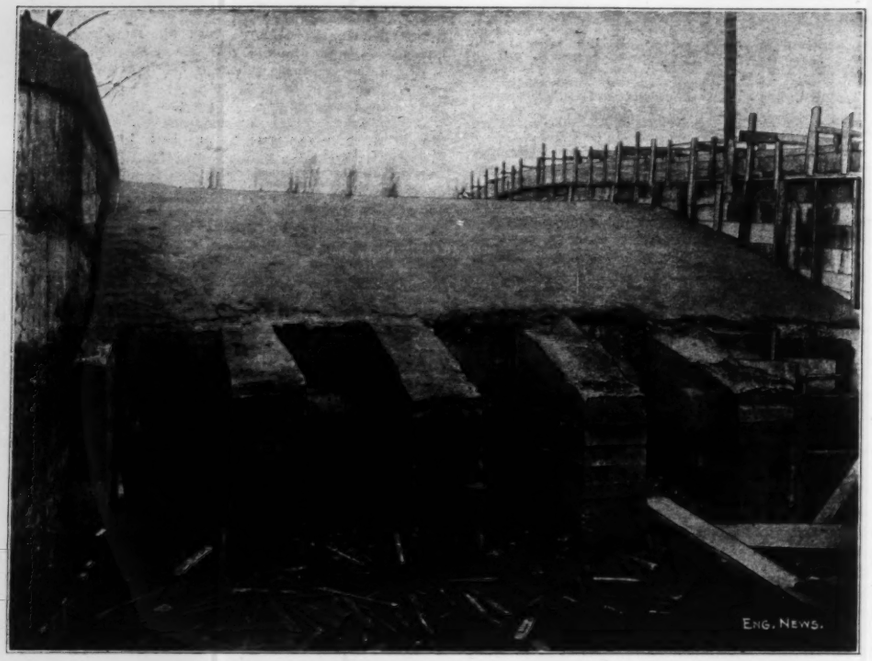


FIG. 2. VIEW AT BACK OF ABUTMENT DURING CONSTRUCTION, SHOWING DEFICIENT SUPPORT OF BUTTRESSES.

now being erected in a park in one of our large cities is shown in the accompanying drawings and views. The bridge, shown in section in Fig. 1, is an arch of 75-ft. span reinforced near both surfaces. It is intended to be supported on ribbed abutments resting on piling. The abutments are, however, entirely too light to support the thrust of the arch, even when the earth fill around the abutments is taken into account. The ribs have no support at the back and the view Fig. 2 shows how a stick of wood was inserted for 2 ft. at the base of one of the ribs. All of the ribs were built in this same manner with little support at the rear where the greatest thrust from the arch will be exerted. The designer might have taken advantage of the resistance of the piling to lateral thrust by reinforcing the back edge of the ribs continuously, but instead there is a break in the reinforcement, and failure will likely result by cracking of the abutment at the point marked A, after which the lower part will overturn, permitting collapse of the arch. Here is a striking example of the necessity for reinforcement along the outer surface opposite the springing instead of along the inner surface as required by the elastic theory. The reinforcement shown at B is useless, whereas had it been used at A instead it might have saved the design.

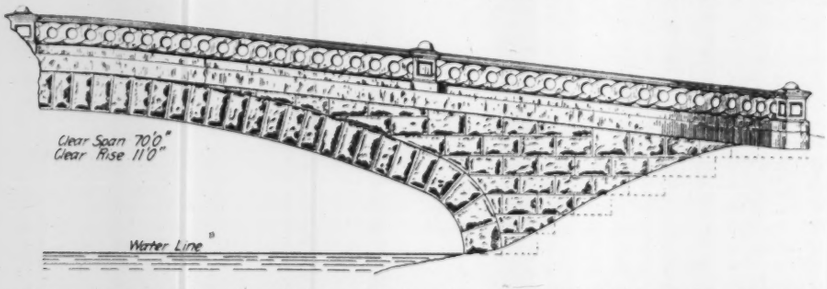


FIG. 3. HALF SIDE ELEVATION OF ARCH, SHOWING IMITATION OF STONE WORK.

While this is an extreme case of an abutment too light to support the thrust, yet it illustrates the error in the fixed tangent assumption, since it is not economy in any arch to build an abutment that has sufficient mass to be absolutely fixed.

The bridge described contains many other errors of design, both ornamentally and structurally. The spandrel wall, although of concrete, is blocked off to imitate stone (Fig. 3), and the arch ring is marked off to uniform thickness for its entire length. To enable this depth to appear at the crown the earth fill and roadway is increased in depth to nearly 3 ft. over an arch 16 ins. thick. This adds to the load at the crown, increasing the thrust against the abutments, which are poorly qualified to resist it by reason of the decreased fill above them. The designer has failed utterly to appreciate the fact that he was working in reinforced concrete and has attempted to give the effect of a stone arch instead, when for lightness and graceful lines reinforced concrete lends itself far more readily to arch construction than stone. The roadway is heavily crowned, with no apparent excuse, since the crown of the arch rises over 5 ft. above the level of the approaches. The only plausible reason for this awkward rise is the attempt to imitate a stone arch.

The curve of the arch is moreover not properly selected to add to the stability of the abutments. The arch has an extremely heavy load at the crown, yet its curve is very full at the springing; as a result the thrust of the arch is very high against the abutments, already too light to withstand their overturning moment. If this structure stands when the centers are struck, it will be due to the fact that the material of which it is built is far better adapted to such a structure than is the material imitated. With the exception of the lack of bearing for back of abutments, the work has been well executed in construction.

Very truly yours,

Daniel B. Luten.

Indianapolis, Ind., Nov. 15, 1907.

[We must observe that the objections made in the above letter to the use of the elastic theory in arch design have no direct relation to the subsequent criticism of a specific arch. The axiom that an arch must have substantial abutments to receive the thrust and carry it to the foundations is not changed by questions of elastic theory or pressure-line method. In every case, it is certain that the thrust exists, and usually the thrust is not far distant from the general line of the arch. To receive this inevitable thrust at the top of a narrow abutment block is always questionable, and the ability of backfill behind the abutment to offer rigid support against the arch thrust is highly conjectural. It would seem, therefore, that Mr. Luten's objections to the application of the elastic theory can hardly seek a proof in the example cited.

It is by no means clear, to us at least, that the elastic theory enjoys enough of either use or abuse in arch design to make the questi... very serious. The three-hinged assumption ... still much in favor, and there are not a few des...ers, perhaps, who are willing to accept an arch ...sign so long as a pressure line can be passed ...*where* through the arch ring.

The statements contained in the above l... as to the assumptions involved in the use of t... lastic theory, and the statement that the ...stic theory shows tension at the intrados n... the springing, can also not be accepted offhand. ...]

The Lacing of Compression Members.

Sir: In an article on "The Lacing of Com...ion Members" in your issue of Nov. 7, 1907, some a...mptions are made which are not warranted:

1st.—It is assumed that the increments, v, p... split length of column (by which the lattice bars tran... the stress, V, from one leaf or channel to the othe... are equal. This is not so stated specifically but it is ...wn by equation (1) and by the application made in ... example. This assumption is true only when the m...ent diagram is a triangle. If it is sinusoid, which is th... ...e for an ideal column, the maximum increment ... ...gth (about 1.57) times the mean; if it is a parabola the ...aximum is twice the mean. For a column with pivoted ...ds, in which the imperfections in material and workm...ship inevitable in practice result either in an unintention... and equal eccentric application of the loads at both end... and to the same side of the true axis, or in a nearly circular curve in the true axis, the moment diagram will ha... a shape between a parabola and a sinusoid. For the pur...pose of devising a rule for proportioning lattice bars, it is safest to consider it a parabola.

2d.—The formula

$$S = 16{,}000 - 70\,\frac{l}{r}$$

in which S = average allowed unit stress on column, l = total length of column and r = radius of gyration, is applied to the determination of stresses in the lattice bars of a column with rigidly fixed ends. This formula is taken from the American Railway Engineering and Maintenance of Way Association specification, in which it is given for general use without reference to the character of the end connections. Engineers generally concede that a column with rigidly fixed ends will carry more load than one with pivoted or pin ends, but they frequently will not permit any of the compression members of a bridge to be proportioned as if the ends were rigidly fixed for the reason that the ends are not so fixed. In fact in some cases where the ends have an appearance of being fixed, adjacent members by deflecting in opposite directions may take a curve similar to that of columns with pivoted ends. A comparison of the formula in question with the results of experiments on columns with pivoted ends shows that it is a fairly safe one for such a condition; to proportion the lattice for twice the stress which a column with pivoted ends might have is a waste of metal.

By a curious coincidence the errors in the two assumptions balance each other in the example, but in other cases the application of the first assumption without a counter-balancing error might result in weak lattice bars.

3d.—It is assumed that the term $70\,\frac{l}{r}$ in the formula indicates the half difference in the intensity of the mean stress in each channel or leaf, whereas it should represent the half difference in the intensity of the stresses in the extreme fibers.

4th.—It is assumed that "the load at the ends is applied equally to the two leaves." This is not in accord with the fact that columns are liable to unintentional eccentricity in the application of the load. It is probable that if the provision for flexure, implied by the column formulas in ordinary use, was needed in any column, part of the flexure would be produced by eccentric application of the load. It is safer in most cases, however, in proportioning lattice bars to assume that the entire flexure is due to the eccentricity caused by an initial curvature in the axis plus secondary stresses from deflection.

The straight line formula quoted in the article referred to, though not the best that can be devised, is a good one for practical use, and if the term $70\,\frac{l}{r}$ contained therein is provisionally accepted as the proper allowance for flexure it results in a very simple formula for the shear in lattice bars.

Let A = the cross sectional area of the column in square inches.

M = the maximum bending moment in inch-pounds.

v = the half width of the column.

S_m = the maximum transverse shear in pounds when the moment diagram is a parabola.

I = the moment of inertia.

$$M = \frac{I\,70\,\frac{l}{r}}{v} = \frac{Ar^2\,70\,\frac{l}{r}}{rv} = \frac{S_m l}{4}.$$

$$\therefore S_m = \frac{280\,Ar}{v}$$

(This rule and others similar, except for the numerical coefficients, have recently been devised independently by a number of engineers, and one similar, except for the numerical coefficient, was given by Prof. J. B. Johnson, in 1886, in the first edition of "Modern Framed Structures," p. 255.)

For columns composed of two channels with flanges turned out, r/v can be taken as 5/7 without serious error; then, for this assumption $S_m = 200\,A$, and the combined stress in the lattice of one panel, for 45° lattice, is $283\,A$ and for 60° lattice, $231\,A$.

If reasonable assumptions are made as to the limits of the injurious variations from ideal conditions, inevitable in practice, the corresponding stresses in the lattice bars, as determined by analysis, will not be the same for all length of columns; hence the foregoing rule is theoretically faulty. It is, however, a good empirical rule and a safe one, except for very short columns with abutting ends and columns which are intentionally loaded so as to produce a bending moment or which have a large bending moment transferred to them from a connecting member (a floorbeam, for instance).

The stresses to which the lattice bars of columns with imperfect butt joints are liable were discussed by the writer in an article which appeared in your issue of Oct. 3, 1907. Equation 35 of that article indicates the stresses when the cross-sectional area of the lattice bars is known. For the purpose of designing new short columns with properly proportioned lattice it may be assumed in advance: that the stress per square inch of gross sectional area of the lattice bars will be about 10,000 lbs., that the modulus of elasticity is 29,000,000 lbs. per sq. in., and that the tangent of the angles "a" (that is, the equal and opposite angles made in their free state by the abutting faces at the ends of the middle column of a line of three equal and symmetrical columns) is 0.003. Substituting these values in equation (35) of the writer's article and reducing, gives;

For 45° lattice:

$$S_m = \frac{238{,}000\,A}{l^2/r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

For 60° lattice.

$$S_m = \frac{216{,}000\,A}{l^2/r^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

The combined stress in all the lattice in one panel on both sides of the column, for 45° lattice is $\frac{336{,}000\,A}{l^2/r^2}$ and for 60° lattice, $\frac{249{,}000\,A}{l^2/r^2}$.

These equations indicate very high stresses as possible in the lattice bars, and equation (19) of the same article indicates a high stress as possible in the channels when l/r is very small; a fact which should be remembered when considering the superiority claimed for the Forth over the Quebec bridge on account of the larger outside cross-sectional dimensions of its members.

When a column is intentionally loaded in a way which will produce a large bending moment, or a large moment is transferred to it from a connecting member, for instance a floorbeam, the determination by analysis of the possible stresses in the lattice from a combination of such a moment with the moment from column imperfections is such a complicated problem that it is difficult to devise any satisfactory rules. The following, however, is suggested as preferable to a simple guess:

Let p = the mean stress per square inch allowed in a column with the same length and radius of gyration as the given column but without any intentional or transferred bending moment.

A = the area required for such a column.

A' = the additional area required for the intentional or transferred bending moment; determined on the basis of unit stress "p."

S' = the additional transverse shear corresponding to the intentional or transferred moment.

$$S_m = 280\,A\,r/v + \frac{16{,}000\,S'}{p}.$$

The reason for multiplying S' by the factor 16,000/p is to provide for a further shear which will be developed by the deformation of the column.

This formula should not be used when it gives a smaller result than equations (36) or (37), as the case may be.

For a further discussion of this subject the reader is referred to the December Proceedings of the Engineers' Society of Western Pennsylvania.

Henry S. Prichard.

Beaver, Pa., Nov. 8, 1907.

Sir: You are entitled to the thanks of every designer of laced columns for publishing Prof. Morris's paper on "The Lacing of Compression Members" (Eng. News, Nov. 7, 1907, p. 487).

The formula for the increment of stress per unit length of column (transmitted by the lacing from one leaf to the other) $v = \frac{2\,c\,A}{r}$ for square ended column, or $v = \frac{c\,A}{r}$ for pin ended columns, in which c is the constant of a straight-line column formula, is so simple as not to discourage the hurried engineer and is well on the side of safety, for no column, or part of column between lacing, fails except by bending.

In fact n, the half width of column, might better be used in place of r, as it is only the extreme fiber that gets the maximum load or that allowed on shore compression pieces, and as r does not differ much from the distance of center of gravity of leaf from center of column, or of pitch line of lattice from column center.

The formula here proposed

$$v = \frac{2\,c\,A}{n}$$

gives a slightly excessive value of v for still another reason—the connection of the lacing between center of gravity of leaf and the most compressed fiber; but till some one publishes a better one it seems worth using. It changes the designing of lacing from guess work to a certainty of safety.

The application of Prof. Morris's form of the formula to the Quebec bridge chord A9 gives stress in lacing angle of 82,500 lbs. when the chord is deflected enough to make the unit-stress at center of gravity of one leaf 21,000. The form now suggested gives 82,500 × 20.4 ÷ 28¾ = 58,600 lbs. when unit-stress at extreme fiber is 21,000. Of this stress in the lacing 80% is transmitted to the outer rib, giving 23,440 lbs. for each rivet, just about that shown to exist Aug. 28th on account of bending. (Eng. News, Oct. 3, p. 366, corrected by p. 449.)

Adding at least 5,000 lbs. for stress due to direct compression we have 28,000 lbs. per rivet, more than enough to produce slip. Had the riveting been adequate to prevent slip there would have been less bending and probably no collapse.

BRACING OF PONY TRUSS CHORDS.—Reasoning similar to that of Prof. Morris seems applicable to the bracing of top chords of pony truss bridges or the top flanges of plate girders, making the braces capable of resisting such transverse forces at their tops as would, if applied to the chord, produce maximum unit-stress equal to the difference between the allowable unit stress on chord full length of bridge and on chord of one panel length. For the straight-line formula

$$S = S_1 - c\frac{l}{r}$$

$$s = S_1 - c\frac{L}{r}$$

$$S - s = -\frac{c}{r}(L - l)$$

This fiber-stress $-\frac{c}{r}(L-l)$ could be produced by a system of transverse forces increasing toward the bridge center as ordinates of a circle or a parabola and limited though not entirely neutralized by the stiffness of the floor beams and braces.

A simple rule, and at least better than the usual guess, would be to calculate the uniform load required to produce the fiber-stress $-\frac{c}{r}(L-l)$; divide that load by the number of braces, and design each brace to carry its portion of the load with deflection not exceeding that of the unbraced long chord at its center.

If any one of your readers will give a more reasonable theory, he will benefit many, as this may not be the simplest or best possible.

John Bartholomew.

New York, Nov. 10, 1907.

Notes and Queries.

C. H., of Shanghai, China, asks to be referred to experiments bearing on the relative strength of buckled plates laid with the buckle upward and the buckle down. A similar question was asked by a correspondent in our issue of Jan. 26, 1899. Replies by several correspondents in our issues of Feb. 2, and Feb. 9, 1899, show the following:

(1) Tests by Rumschoettel, given by Winkler in "Vorträge über Brückenbau," indicate that buckle plates laid with the convex face down are 3.7 to 4.4 times as strong as when laid with the buckle up.

(2) Tests were made in a Pennsylvania city to guide in the design of a bridge floor, in 1895. Four ¼-in. plates, each having three buckles 39 ins. long by 44½ ins. wide, 3 ins. high, were laid on an I-beam frame and were covered with concrete 4 ins. in least depth. One set of two plates was merely laid on the beams, the other set was bolted with ⅝-in. bolts 9 ins. apart. In each set one plate had the buckles down, the other the buckles up. In both sets, the plate with buckles down gave the higher resistance, both elastic and total; actual rupture was not produced, as the deflections became so great as to bring the plates down on part of the supporting framework. The plates with buckles down were not distorted much out of their original shape, but the plates with buckles up showed bends in the sides of the buckles after failure. The superiority of plates laid buckles down is clearly shown by the tests. No strengthening effect of the bolting is shown by the results.

DIFFICULTIES WITH FLOWING CLAY SUBSOIL AT THE LA BOCA DAMS OF THE PANAMA CANAL.

Difficulties at the La Boca dams are described in a report of Lt.-Col. G. W. Goethals, Chairman of the Isthmian Canal Commission, to Secretary-of-War Taft under date of Dec. 9, printed in "The Canal Record" of Dec. 25, 1907. These difficulties led to a new study of the Pacific end section of the Panama Canal, to determine the relative advantages of the several available locations for the Pacific locks. Four projects were developed as follows: (1) Existing project, with one lock at Pedro Miguel and two locks at La Boca, a 55-ft. level being maintained between Pedro Miguel and La Boca; cost, $58,395,500. (2) Two locks at Pedro Miguel and one at La Boca; cost, $58,467,000. (3) One lock at Pedro Miguel and two at Miraflores, with a 500-ft. sea level channel between Miraflores and La Boca; cost, $50,927,300. (4) One lock at Pedro Miguel, one at Miraflores and one at La Boca; cost, $66,690,600. In each case all locks and dams are founded on rock. Project (1) is the location recommended by the minority of the Board of Consulting Engineers early in 1906, and adopted, against the suggestion of Secretary Taft that the La Boca locks would be too much open to damage or destruction by the fire of a hostile fleet. When the building of the dams was begun, the material on which the fill was dumped proved so soft as to make it necessary to decide on rock foundation. This, however, increased the expense to the figure of $58,395,500, as above given, and made the Miraflores location for the two lower locks preferable. Lt.-Col. Goethals therefore recommended the abandonment of the location described by Project (1), and the adoption of Project (3). President Roosevelt has approved this recommendation, as noted briefly in our last issue (p. 21).

The unexpected difficulties encountered at La Boca are thus described by Lt.-Col. Goethals:

> To construct the dams [the three dams surrounding the La Boca locks—Ed.], trestles were built along the toes of the Sosa-Corozal dam, from which to dump material from the Culebra Cut. The trestles failed, after the dumping of the material from them began, and the material overlying the rock moved laterally carrying the superimposed mass with it, the dumps flattening out until the side slopes were about 1 on 12 and even less. In places this lateral motion continued for two weeks after dumping had stopped. The ground on either side of and at some distance from the dump was forced up forming mounds of mud, the crests of which gradually approached the level of the top of the dump proper. After an equilibrium was established between the dump and the adjacent mounds, the hump or wave would again move out when the track was shifted towards it, accompanied by a sudden vertical settling of the track of six to ten feet when loaded trains were applied. It was thought that if the trestle were lower, better results might obtain, but with the west trestle only 8 ft. above the surface of the ground and the piles driven to rock, similar failures followed.
>
> When the difficulties on the east trestle developed, an investigation, by boring and test pits, was made to determine the character of the material overlying the rock. It was found to be for the greater part an unctuous blue clay without grit, possessing but little supporting power, instead of stiff clay as indicated on existing profiles.
>
> The investigations clearly demonstrate that the construction of dams which would remain in place after the lake is filled can be accomplished only by the removal of all material overlying the rock, and, after preparation of the rock surface, by building the dams of selected material. The depth to which the material will have to be removed is from 10 to 70 ft., and suitable material for dam construction will have to be transported to the site. Investigations of the foundations of the Sosa-San Juan dam show that the material is the same; the difficulties to be encountered will be greater because of the river.
>
> It is estimated that the cost of building the two dams, following the plan that the investigations and work done indicate as necessary, will be about $11,573,800. The estimate submitted by the minority of the Board of Consulting Engineers for these dams is $4,314,000, a difference of $7,259,800.

A SPEED OF 44 MILES or 38.3 knots per hour was attained on Dec. 17 over a mile course by the British torpedo boat "Tartar." The "Tartar" had recently broken the record for boats of her class, with a speed of 34.7 knots per hour over a two-hour trip, and a record mile at 36 knots per hour. (Eng. News, Dec. 12, 1907, p. 648.)

THE ANALYSIS OF CONCRETE.

By ROYAL L. WALES.*

The necessity for the critical examination of concrete in paving, reinforced structures, etc., should be, and usually is, precluded by a proper testing of the materials used, and a rigorous inspection of the work as it progresses. It sometimes happens, however, that through the inefficiency or connivance of those entrusted with the supervision, work of a very inferior grade is allowed to be completed. In cases open to suspicion it first becomes necessary to establish the fact of inferiority, and then to locate the cause, in order that the blame may be rightly placed and suits for damages successfully conducted.

The methods commonly employed in the determination of such defects depend largely on hammer and pick tests, combined with visual examination of the specimens for improper mixing, use of dirty or clayey material, insufficient tamping, etc. Such methods, while they invariably establish the fact that a given concrete is good or bad, as the case may be, do not always point out the specific reason for such a quality. For example, it would be impossible to tell by such tests whether the low strength of a certain concrete were due to an improper proportioning of the ingredients, or to the use of an inferior cement. In other words, it could not be told whether the blame should be placed with the contractor for using an excessive amount of inert material, or with the testing department for allowing a poor cement to be used.

A condition of affairs in which all these considerations came up, recently arose in a certain southern city. Contracts for the paving of the city streets were let to three different concerns engaged in such business. The materials used by these contractors were inspected and tested by the city engineer, and allowed by him to pass. The work was supervised by inspectors in the employ of the city. In spite of these measures, protests soon began to arise from the citizens that the work was far from satisfactory. In this event, an expert was employed to investigate the matter. As his report was unfavorable to the contractors, it was decided to corroborate either the statement of the city engineer or that of the expert mentioned, by a second and very thorough investigation by two other engineers.

These engineers made the usual hammer and inspection tests, but wishing to secure further data on which to base their reports, secured samples of the concrete paving in various parts of the city and submitted them to the writer for a determination of the proportions by volume in which the cement, sand and rock were mixed.

The problem resolved itself into two parts: a determination of the ratio of cement to sand in the mortar, and that of the ratio of mortar to rock. Suggestions of determining the cement by lime and silica were vitiated by the fact that a natural cement of somewhat uncertain composition had been used, and that a limestone rock constituted the coarser part of the aggregate. The sand used was river sand obtained by dredging.

RATIO OF CEMENT TO SAND IN MORTAR.—Samples of the sand were obtained at different times and places. Such samples, treated with hydrochloric acid and digested to dryness, showed an insoluble residue of 96.5%. Various natural cements obtainable showed under similar treatment an insoluble residue of from 21 to 29%. The difference in the solubilities of the sand and the cement was made the basis of their determination. The process in detail was as follows, being practically a standard silica determination.

About 1½ grams of the dried mortar was accurately weighed into a casserole, covered with 10 cc. of strong hydrochloric acid and digested to dryness on the hot plate. The residue was then moistened with acid, an excess of water added, and the solution filtered. The residue and filter were washed successively with dilute acid and water, after which they were transferred to a weighed crucible and ignited. The per cent. insolubility of the mortar was then calculated from the ignited residue. From the insolubility the ratio of sand to cement was determined as follows:

If X = parts of sand in 100,
then $0.965\ X + (100 - X)\ .25 = P$ = per cent. insolubility.

Blanks were made up, not only with sand and fresh cement, but also with sand and crushed briquettes of neat cement which had set for the same length of time as the cement in the paving to be tested. This was done to ascertain if the solubility of the cement was affected by any reactions which might have taken place with time. The proportions of the blanks were determined as accurately as possible on laboratory scales. Following are listed some of the blanks and their proportions by analysis.

As made up.	% insoluble.	Ratio calculated from above equation.
1 fresh cement to 2 sand..	73.0	1 to 2.04
1 fresh cement to 3 sand..	78.9	1 to 3.02
1 ground briquette to 2 sand	73.4	1 to 2.08

These results were obtained using the average values for the insolubility of the cement. Of course, if the actual values had been used the checks would have been closer. If the lowest and the highest values for insolubility of the cement had been used instead of the mean, the results for the 1 to 2 mixtures would have been respectively 1 to 2.22 and 1 to 1.88. It will thus be seen that, although the variable silica content of the cement introduces an element of uncertainty, this is of little importance, since using a mean value, in no case would it be possible to mistake a 1 to 2 mixture for a 1 to 2½, or the latter for a 1 to 3 mixture.

RATIO OF MORTAR TO ROCK.—In determining the ratio between the mortar and rock, two entirely distinct methods were used. The results of these methods were checked against each other and against blanks.

By the first method, the sample was thoroughly dried and the rock separated from the mortar by attrition. The weights of the mortar and the rock formed the basis for the calculation of their proportions by volume. In this process great care had to be exercised to remove the mortar completely from the rocks, and at the same time not to pulverize the soft rock.

The other method depended on the fact that the specific gravities of the rock and of the mortar (including the voids in the latter) were quite distinct. The specific gravity of the rock used was 2.66. That of the mortar varied from 1.57 to 1.82, depending on the proportions and on the amount of compacting. On account of this variation of the specific gravity of the mortar in the different samples, it was necessary to determine it for each case. The method in detail was as follows:

A specimen concrete block weighing from 20 to 30 lbs. was taken to get a fair representation of the whole, and to allow of sufficient percentage accuracy in the weighing, and by the usual immersion method its specific gravity was determined. For finding the specific gravity of the mortar, a smaller sample was used, one that could be weighed on more delicate balances. The loss of weight of the lump in water after saturation gave its exact volume. The rock was then carefully separated from the mortar (which could easily be accomplished on so small a sample) and weighed. The weight of the rock divided by its specific gravity gave its volume. The difference between the volume of the lump and that of the rock, gave the volume of the mortar. The difference between the weight of the lump and that of the rock gave the weight of the mortar. The ratio of the volume of mortar to its weight gave its specific gravity (metric system).

Knowing the specific gravity of the whole sample and of its two constituents, a simple calculation gave the proportion of each. For example, letting X represent the volume of solid rock per cu. ft. of concrete, 2.66 the sp. g. of the rock, 2.20 that of the concrete and 1.73 that of the mortar,

$$2.66\ X + (1 - X)\ 1.73 = 2.20$$

from which X or the cu. ft. of solid rock per cu. ft. of concrete, appears as 0.529. With [illegible] in the rock at 51%, this corresponds to 1.0[illegible] cu. ft. of unpacked rock per cu. ft. of concrete.

The amount of mortar per cu. ft. of [illegible] was found to be 0.471 cu. ft. This multiplied by the weight per cu. ft. of water, by the specific gravity of the mortar, and by the [illegible] per cent. dry mortar (the moisture having been determined), gives the weight of mortar per cu. ft. of concrete. Knowing the ratio between the cement and sand from the chemical analysis, in this case 1 to 4, the weight of the cement and therefrom its volume were ascertained.

The weight of mortar per cu. ft. of [illegible] came out for this sample 48.5 lbs. The [illegible] in this mortar then weighed 9.7 lbs. The cement weighed loose 7.15 lbs. per cu. ft. There were, then, 0.136 cu. ft. of cement per cu. ft. of concrete.

From this data it appears that the proportions of this particular sample by volume were 1 to 3.8 to 7.95.

Determinations by both the above methods gave, considering the inherent difficulties of the task, very concordant results, some of which are listed below.

Parts cement.	Parts rock by separation.	Rock by sp. g.
1	5.25	5.00
1	4.50	4.81
1	6.25	6.45

The work outlined above demonstrates the feasibility of analyzing with all necessary accuracy even so uncertain a concrete as that of natural cement. Of course that of Portland cement would be even more positive as the silica content of the latter is practically constant. While not necessary in all cases of investigation, such analyses may be of very considerable importance in such circumstances as indicated above.

*Asst. Professor of Experimental Engineering, University of Tennessee, Knoxville, Tenn.

NOTES ON THE WATER-WORKS OF LONDON AND LIVERPOOL.

The works of the various water companies supplying London and vicinity were recently taken over by the Metropolitan Water Board, in accordance with an act of Parliament passed in 1902. The works thus consolidated under one management comprise—in point of population supplied, although not in the volume of water furnished—the largest water-works undertaking in the world. Some interesting facts regarding these works are printed below, as taken from the report of the Metropolitan Water Board for the year ending March 31, 1906. The indebtedness of the board, mostly purchase money, was about $213,000,000 on April 1, 1906. Nearly all this debt bears 3% interest, and most of it (some 7-11) is "redeemable at par at option of board after expiration of 30 years from date of creation," and most of the remainder is likewise redeemable at the end of 60 years.

The total population supplied is 6,791,000, and the average daily amount supplied in 1905-06 was 38.77 U. S. gals. per capita, or 252.6 U. S. gals. per service. About 95% of the 1,043,293 services are under "constant supply," and all of the services will be supplied throughout the 24 hours as rapidly as necessary changes in the works of the old companies can be made.

The water drawn from the Thames and Lea is filtered through 142 beds, with a total area of 145.16 acres, or .67-acre per Imp. gal. (.56 per U. S. gal.) per acre daily.

Prior to the acquisition of the various works by the board, the water supplied by the several companies had been subjected to periodical examination for many years. On July 21, 1905, after a public advertisement which brought out 38 applicants, Dr. Alex. C. Houston was appointed Director of Water Examinations. Many of our readers will remember that Dr. Houston has been prominently connected with the sewage treatment investigations of the London County Council and of the Royal Commission on Sewage Disposal. The salary of the Director of Water Examinations has been fixed at £1,000 a year, under a strict agreement for the whole time of the ex-

assistant, who, in the language of the report, is "not to be directly or indirectly engaged in any other profession, business or occupation." A central laboratory has been established, at which there are employed under Dr. Houston 2 chemists, 2 bacteriologists, 2 laboratory assistants, 2 clerks, 3 collectors of samples, and 3 laboratory boys.

For the six months ending April 30, 1906, 5,533 samples of water were examined either chemically or bacterially. The filtered water from the Thames for the period showed an average content of 11.2 bacteria per c. c., or only 8 per c. c., exclusive of samples showing more than 100 bacteria per c. c. Of the 9 main supplies making up the total, the lowest average total bacterial filtered water content was 3.2 per c. c., and the highest 25 per c. c. The total bacterial reduction, by subsidence and filtration, was from 859 to 6.2 in the New River water, 2,728 to 9.8 in the Lea, and 1,282 to 11.2 in the Thames. The relative percentage of unfiltered and filtered water samples containing B. coli was 83 and 29, respectively, 87 and 18, and 49 and 23, for the Thames, Lea and New rivers; but the unfiltered samples were of 1 c. c., and the filtered-water samples of 100 c. c. Moreover, these figures are for all coli, whereas the reduction of the typical forms of coli was some 20% greater than the non-typical. It may be added that the types of coli are studied and classified in great detail at this laboratory.

Tests for B. enteritidis sporogenes showed negative results, with 10 c. c. of water, in 43.9% of raw water from the Thames, 51.6% from the Lea, and 83.9% from the New River, as compared with 99.92% of the three waters after filtration, together with unfiltered Kent water. The 1-c c., and .1-c. c. samples of filtered water gave negative results on the B. enteritidis sporogenes test.

Curiously enough, while the report contains the names of the 66 members of the board, and also of the clerk of the board, none others are mentioned except incidentally. We are able to state from other sources of information, however, that Mr. Wm. Booth Bryan, M. Inst. C. E., is Chief Engineer of the board.

LIVERPOOL.

The city of Liverpool, in marked contrast with the Metropolitan Water Board, gives prominence to the names of the principal officers of its waterworks, publishing these in considerable detail in the report of Mr. Joseph Parry, M. Inst. C. E., Engineer in Chief.

Mr. Parry's report covers the calendar year 1906. The population "supplied within and around the city (compulsory area)" is 888,066, to which there is furnished an average of about 35.8 U. S. gals. per capita per day. In addition, the city supplies the town of Chorley, with a population of 29,000, and it also supplies, in bulk, 16 "towns and cities," bringing the total population supplied up to about 1,080,000.

The per capita consumption of 35.8 U. S. gals. supplied in the city and immediate surrounding districts is divided as follows: For trade purposes, by meter, 9.4 gals.; for ships, by meter, 0.9; for workhouses and industrial schools, by meter, 0.9; for miscellaneous purposes, by assessment (quantity estimated), 2.2; for public purposes, including flushing sewers and drains, watering and washing streets, public baths and washhouses, fountains, cattle troughs, urinals, etc., 2.2; for extinguishing fires, 0.03; for domestic purposes, hotels, public-houses, warehouses, offices, shops, and all waste and leakage from public and private pipes and fittings, 19.9; salt water supplied for public purposes, 0.2; water for condensing at the pumping stations, 0.1; total, 35.83 gals.

Only 145 fires for which water was drawn from fire hydrants occurred in the city and its suburbs (nearly 900,000 population) during the year.

The division of Plumbing and Waste Inspections made a total of 231,435 inspections and reinspections during the year, of which 183,050 were "after night reports," indicating waste or leakage. Of 38,295 defects found, 18,777 were repaired by the inspectors at the time of inspection. During the year, 73,247 fittings were tested [presumably new fittings not allowed to be put in place before testing].

The filtered water supplied Liverpool showed an average of only some 20 bacteria per c. c. for 365 samples collected at two taps in the city. The examiner, Prof. Sir Rubert Boyce, F. R. S., reported that

> Bacillus coli communis, Bacillus enteritidis sporogenes and other specific organisms were not present on any occasion.

There are now no residences on the Vyrnwy drainage area except those of "a few farmers and their laborers." Housing has been provided, according to a statutory requirement, for all dispossessed members of the "laboring classes."

During the past half-dozen years, considerable attention has been given to the possibility of so treating the Vyrnwy supply as to lessen the deposits in the pipes which carry the water to the filtration plant. It was found that promising results could be obtained by passing the water upward through fine copper gauze screens, if a sufficiently large screening area was provided to keep the velocity low; but a "much greater degree of improvement was effected by passing the water through layers of coarse sand or gravel." Surveys were made with the idea of providing "strainer beds" sufficient to filter the water at the rate of 18,000,000 U. S. gals. per day. These beds have not yet been built, and need for them has been made less urgent by the completion of a second pipe line, which renders it much more practicable to clean the conduits by passing through them, as has already been done in the past.

Some 40 pages of the report are taken up by appendices devoted to forestry operations on the Rivington and Vyrnwy drainage areas. A large number of plantations and old woods, and also several nurseries, are maintained, and altogether some hundreds of thousands of trees have been planted. The areas planted, or to be planted soon, run into many hundreds of acres.

THE TEACHING OF MATHEMATICS FOR ENGINEERING STUDENTS.

At the recent annual meeting of the American Association for the Advancement of Science, held at Chicago the last week in December, special sessions were held to discuss the teaching of mathematics to engineering students. These special meetings were arranged jointly by the Association and the American Mathematical Society.

The following summarizes the principal points in the papers and discussions:

Prof. E. J. Townsend (University of Illinois) referred to the tendency to increase the requirements in mathematical work for entrance to colleges. In an examination at the University of Illinois, 40% of the class failed in elementary algebra, and 23% of these failed at a second examination a few weeks later. He advocated more attention to geometry as a practical study, and this view was endorsed by several other speakers. There is a growing demand for a better correlation of mathematics with general education, and he would eliminate non-essentials for engineering students, but would not teach any mathematical principles empirically or allow students to disregard the necessity for demonstrations. In revising the scheme of education, the general student should have more mathematics, rather than the engineering student have less. Mathematics should be taught by mathematicians, and engineering by engineers; if the mathematician wants to teach engineering he should be familiar with it, and the engineering instructor should review and revise his knowledge of mathematics from time to time.

Prof. Alexander Ziwet (University of Michigan) dealt with the practice at a number of foreign universities, where mathematics is given a higher position than in this country. He saw no need of special mathematics for the engineer, or of special methods for teaching mathematics to engineering students.

Mr. Charles F. Scott (Consulting Engineer for the Westinghouse Electric & Mfg. Co.) quoted a prominent engineer as saying that the ordinary engineer does not use higher mathematics in his work because he does not know how. A teacher who is simply a mathematician will teach his subject in such a way that it shows the student no practical use or application of what he is taught. What is needed is a combination of mathematics and common sense, so that when a problem is presented the student will have the ability to grasp it and to realize all the elements that enter into it. The gist of his view in the matter was that mathematics should be, but is not, a familiar working tool of the engineer.

Mr. R. S. Woodward (President of the Carnegie Institution) remarked that mathematics may be taught at too early an age, before the student's logical faculty is developed. He suggested that text-books should have plain statements of facts and principles, without demonstrations. Engineers are often inclined to look down upon mathematical workers, while the teacher of mathematics is apt to be narrow in his ideas, failing to realize the practical relation of mathematics to engineering. Again, in many cases, men of inferior accomplishments and without any practical knowledge or experience outside of the class room, are employed to teach mathematics to the students. In this country the skilled mathematician is not yet appreciated at his real value.

At the meeting on Dec. 31 Prof. Gardner S. Williams (University of Michigan) remarked that those who teach engineering and those who teach mathematics to engineers should both practice engineering. As a matter of fact there are very few engineers who are competent to teach higher mathematics. It is not so much a problem of what to teach, as how to teach. He suggested that the methods of teaching mathematics in the grammar schools need overhauling, and that physical science might well be taught in these schools. It has been stated that only a small percentage of the graduates of a certain "so-called" engineering school are employed in positions that require the use of mathematics, but as a matter of fact few of these graduates are properly educated in the subject. Indeed, one of the directors of that institution said that he did not see the use of teaching descriptive geometry to mechanical engineers. Mr. Williams thought too much attention was given to formulas, rather than to the analysis of problems. If engineering schools are content to accept as teachers men who have failed in practice, they have only themselves to blame if unsatisfactory results cause criticism of the institutions.

Prof. F. S. Woods (Massachusetts Institute of Technology) also referred to the low salaries paid to mathematical instructors, and the relation of this to the competence of the men and the results of their work. While engineers are prone to point out what they consider to be defects in methods of teaching, they rarely suggest any remedy, and as a matter of fact their criticisms are often based upon their own personal experiences of 10 or 20 years ago. They do not realize that the teachers see and realize the defects as well as the engineer. After the mathematical teacher has done his work, the professor of engineering should so teach that the student will be obliged to use the knowledge imparted to him by the former.

Prof. A. N. Talbot (University of Illinois) presented a short paper, in which he pointed out that in the effort to avoid barren formulas the teaching may go to the other extreme and waste time on matters of minor importance.

Another speaker claimed that the study of mathematics is not to develop mental faculties but to give men power to do things. He denied that mathematical students were more accurate than others, or had better powers of observation. In very many cases the students who "know" mathematics do not know how to use them. Teaching in mathematics should be continuous and consecutive, from school to college, and should be a compulsory subject. Geometrical methods are used too little, and this is specially true of descriptive geometry.

A paper by Mr. J. A. L. Waddell, which was read in abstract, repeated the statements as to the desirability of ascertaining whether a student really understands and has analyzed a problem or whether he has simply memorized some rules and formulas. He, like several others already mentioned, believed that geometry is of great importance and should be given greater attention.

MINIMUM END-LIFT DEVICE OF A GERMAN SWING-BRIDGE.

A rather novel end-lift arrangement is used on a swing-bridge recently built at Oldenburg, Germany. The arrangement is shown diagrammatically in Fig. 1 herewith, taken from an article in the "Organ für die Fortschritte des Eisenbahnwesens in Technischer Beziehung." The end lifts are designed for a minimum lift, and the amount of lift is limited by upper bearings formed by pairs of lugs projecting from the

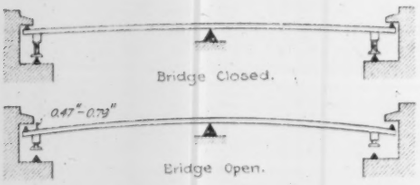


Fig. 1. Diagram of Method of Support and End-Lift, Swing Bridge Over the Hunte at Oldenburg.

abutment and from the end floor beam of the bridge, respectively. The lift is not sufficient to develop an end reaction greater than the possible live-load uplift, and the upper bearing forms a firm anchorage against any resultant uplift.

The original idea of the designer, it seems, was to use no end-lift devices whatever, but to lock the bridge in its fully deflected (cantilever) condition. In this case, of course, the end support must be so detailed as to be capable of taking both downward and upward load from the end-shoe of the bridge. With this construction there is no load in either direction on the end support when there is no live-load on the bridge; the bridge can be swung open without further preparation than withdrawing the latch, and after closing requires no further operation than latching to make it ready for traffic. Since the end supports carry no load, there is no frictional or other resistance at the end to oppose the turning operation. The arrangement should therefore reduce both the time and the power required for opening and closing the draw.

Temperature effects or unbalanced wind pressure may interfere with this ideal operation, however. If, for instance, the top chord of the trusses is warmer than the bottom chord, the bridge tends to deflect downward at the ends, and, consequently, throws a heavy downward load on its end supports. If the bridge required to be opened at such a time, a large extra resistance to turning would be developed by the friction of the shoes on their supports; and on the return movement of closing, the shoes, unless guided into place by inclined approach planes, would catch on the abutment, and in any case would again develop the increased frictional resistance, which might interfere with complete closing.

A modified arrangement, as shown in the sketch, was therefore adopted. It is a combination of the end-lift and the locked cantilever arrangements. An end-lift is used just sufficient to insure that when the lift is released the bridge will clear its end supports under all conditions of temperature, wind, etc. As the lifting force required increases with the lift, a minimum lift is used, in order to keep down the required dimensions and strength of the lift mechanism.

The amount of lift adopted is not sufficient to develop a dead-load reaction so large that no live-load uplift can occur. An upper bearing is therefore provided, just as in the bridge without end-lift, to prevent raising and hammering at the ends. When the bridge is in fully deflected condition the upper bearing is not in contact; the end-lift forces it up into bearing.

The end-lift mechanism employed is shown by Fig. 2. A double-acting screw-jack forms the lifting device, of which there is one under each end of either truss. The support on the abutment is detailed as a rocker-bearing, to allow for free longitudinal expansion. The rocker is held by springs in its central position, when free. The upper bearing is formed by a bracket projecting from the end floor beam, bearing against an inverted bracket on the abutment. The latter is attached to a steel anchorage bedded in the abutment masonry. The upper bearing is also detailed as a rocker bearing. Fig. 3 is a view of the end of the bridge, showing the upper and lower bearing and a jack.

The two jacks at either end of the bridge are operated by bevel-gearing from a cross-shaft attached to the end floor beam, as shown, which shaft is driven by a longitudinal shaft receiving its power from an electric motor at the turntable. The amount of lift varies, of course, with the vertical position of the end of the bridge, and this, of course, depends on the temperature, wind and load conditions; the end is always lifted until the upper bearing is in contact. The jacks have a maximum stroke of 1¼ in., but under normal conditions the clearance at the upper bearing requires a lift of only ½ to 13-16 ins.

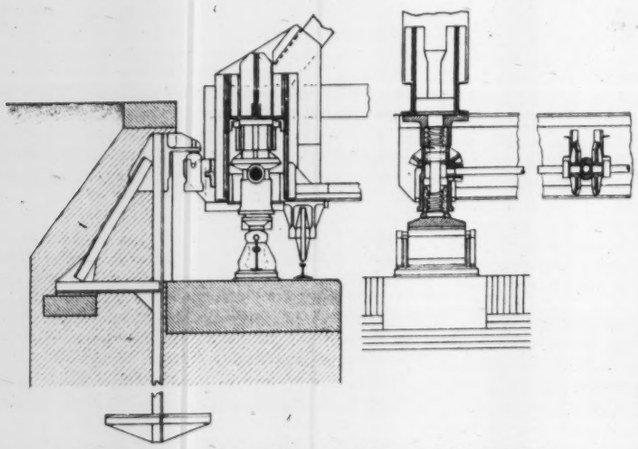


FIG. 2. END-LIFT MECHANISM OF HUNTE BRIDGE.

In order to guide the end of the bridge to proper level, a track-rail is fixed on the abutment, which engages a group of vertical guide-wheels carried by the bridge. The bridge is opened always in the same direction, and the receiving end of the rail is sloped to catch the wheels and guide the bridge to its proper level gradually.

In the matter of design of the turn-table, the bridge is purely center-bearing, but it has a ring of six equally spaced steadying wheels bearing on a circular track of 24½ ft. diameter. The center pivot is detailed as per Fig. 4. The pivot is a forged steel shaft, 20 ins. in diameter, set firmly in an anchorage frame built in the masonry of the pier. The bridge is held to this pivot by two collars—one at the top, fitting tight, and the other 5 ft. lower down, fitting with ⅛-in. play. This lower ring is intended to come into bearing when the steadying wheels deflect too much or break. These wheels are carried by stiff leaf-springs. There is, further, a bearing under the center of either truss, on which the truss seats when the bridge is fully closed. This bearing is designed to take the live-load, and is adjustable in height. The end bearings are also adjustable, to provide against settlement of the abutment masonry. The bridge is turned by a pinion engaging in a quarter-circle rack fastened to the steadying track. The pinion is driven by an electric motor separate from the end-lift motor.

Actual trials showed that the time required for opening or closing, including the end lifting,

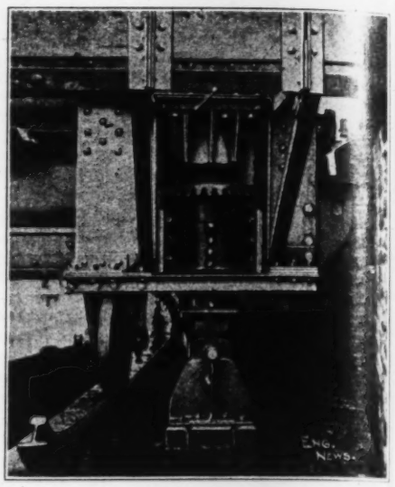


Fig. 3. View Lifting Jack and Upper and Lower End Bearings, Hunte Bridge.

is 85 to 115 sec., depending on the wind. The maximum power draft is about 7.5 KW. (60-70 amp. at 110 volts).

The bridge in question is a double-track railway bridge, 197 ft. long. It replaces two closely adjoining bobtail draws, single track; the new bridge gives a wider channel, substitutes a single passage for two, and gives more clearance above the water. The requirements for the new bridge were: two channel openings of at least 52½ ft. each, 11½ ft. clearance at least above ordinary

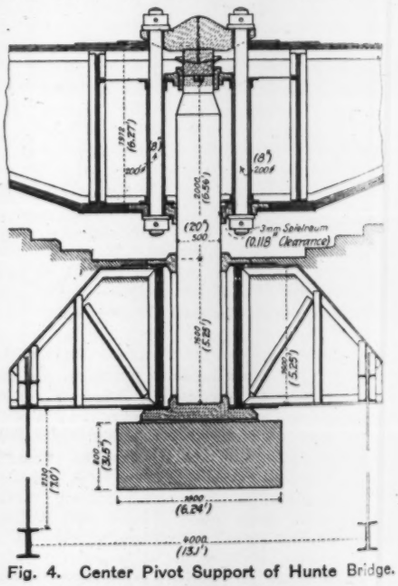


Fig. 4. Center Pivot Support of Hunte Bridge.

low water, floor thickness not more than 36 ins., time required to open or close not more than 2 to 3 mins. The crossing is on a skew, but the bridge is square-ended. It has two trusses, 29½ ft. c. to c., and a sidewalk on either side cantilevered out beyond the trusses. The trusses have a single web system, with diagonals sloped downward from the middle.

The bridge was built by the Gustavsburg works of the Vereinigte Machinenfabrik Augsburg und Nürnberg. The superstructure cost about $30,000.

LUMBER PRODUCTION IN THE UNITED STATES IN 1906.

The growing scarcity and increasing price of lumber in this country makes of particular interest the statistics on lumber production in the United States in 1906, which are given at some length in Circular 122 of the U. S. Department of Agriculture. The information given is based on reports from 22,398 lumber manufacturing establishments distributed throughout the United States. The Bureau of the Census and the Forest Service have cooperated in this work, and have been assisted by various manufacturers and lumbermen's associations and also by the Forest, Fish & Game Commission of New York State. The mill value of the 37,550,736,000 ft. of lumber output of 1906 was $621,151,388. In addition there were produced 11,858,260,000 shingles valued at $24,154,555, and 3,812,807,000 laths valued at $11,490,570. Making allowance for incomplete reports, the total lumber cut for the year was probably 40,000,000,000 ft., of a total mill value of $700,000,000. The lumber production by leading species for the whole country, for the years 1899, 1904 and 1906, is given in Table I.

TABLE I.—COMPARISON OF LUMBER PRODUCTION OF LEADING SPECIES, 1899, 1904, 1906.

Kind.	1899. M. ft.	1904. M. ft.	1906. M. ft.	Per cent. increase (+) or decrease (—) since 1899.
Yellow pine	9,658,923	11,533,070	11,661,077	+ 20.7
Douglas fir	1,736,507	2,928,409	4,969,843	+186.2
White pine*	7,742,391	5,332,704	4,583,727	— 40.8
Hemlock	3,420,673	3,268,787	3,537,329	+ 3.4
Oak	4,438,027	2,902,855	2,820,393	— 36.4
Spruce	1,448,091	1,303,886	1,644,987	+ 13.6
Western pine	944,185	1,279,237	1,386,777	+ 46.9
Maple	633,466	587,558	882,878	+ 39.4
Cypress	495,836	749,592	839,276	+ 69.3
Poplar	1,115,242	853,554	683,132	— 38.7
Redwood	360,167	519,267	659,678	+ 83.2
Red gum	285,417	523,990	453,678	+ 59.0
Chestnut	206,688	243,537	407,379	+ 97.1
Basswood	308,069	228,041	376,838	+ 22.3
Birch	132,601	224,009	370,432	+179.3
Cedar	232,978	223,035	357,845	+ 53.7
Beech	(†)	(†)	275,861	....
Cottonwood	415,124	321,574	263,996	— 36.4
Elm	456,731	258,330	224,795	— 50.8
Ash	269,120	169,178	214,460	— 20.3
All others	486,848	684,526	936,555	+ 92.4
Total	34,787,084	34,135,139	37,550,736	+ 7.9

*Includes Norway pine. †Not separately reported.

Washington led in total output in 1906, the quantity being 4,305,053,000 ft. This was an increase of 201.5% over the cut in 1899. Louisiana, Wisconsin, Michigan and Mississippi followed, in the order named, after Washington, in the amount of output. The average value per 1,000 ft., by species, for the three years already named, was as given in Table II.

TABLE II.—AVERAGE VALUE PER M., BY SPECIES, 1899, 1904 AND 1906.

Kind.	Average value per M. 1899.	1904.	1906.	Per cent. increase since 1899.
Yellow pine	$8.48	$9.97	$15.02	77.0
Douglas fir	8.67	9.51	14.20	63.0
White pine	12.66	14.93	18.32	45.0
Hemlock	9.98	11.91	15.31	54.0
Oak	13.78	17.51	21.76	59.0
Spruce	11.27	14.03	17.33	53.8
Western pine	9.70	11.29	14.01	44.0
Maple	11.83	14.94	15.38	30.0
Cypress	13.32	17.50	21.94	64.0
Poplar	14.03	18.99	24.21	73.0
Redwood	10.12	12.59	16.64	64.0
Red gum	9.63	10.87	13.46	40.0
Chestnut	13.37	13.78	17.49	30.0
Basswood	12.84	16.86	18.66	45.0
Birch	12.50	15.44	17.24	37.0
Cedar	10.91	14.35	18.12	66.0
Beech	(*)	(*)	14.05	....
Cottonwood	10.37	14.92	17.15	65.0
Elm	11.47	14.45	18.08	58.0
Ash	15.84	18.77	24.35	54.0
Larch	(*)	(*)	11.91	....
Hickory	18.78	23.94	30.42	61.0
Sugar pine	12.30	(*)	16.11	31.0
Tamarack	8.73	11.39	15.63	80.0
White fir	(*)	(*)	12.91	....
Walnut	36.49	(*)	42.25	16.0
Tupelo	(*)	(*)	14.13	....
All others	18.25	20.30	22.20	21.7
Average	$11.08	$12.76	$16.60	49.0

*Not separately reported.

A MINE EXPLOSION in the Bernel Coal Mine at Carthage, N. Mex., was the cause of the death of nine miners and the injury of five others on Dec. 31, 1907. It is supposed to have been caused by the dust and gas combinations which were responsible for the four other great disasters of December, which we have mentioned in this column and in editorials.

AN EXPLOSION in the shops of the Rochester Fireworks Co., at Rochester, N. Y., on Jan. 2, caused the death of two of the women employees. The cause of the explosion has not been discovered. On Jan. 1 the explosion of a converter in the Edgar Thompson plant of the United States Steel Corporation at North Braddock, Pa., wrecked the converting mill, killed two of the workmen and injured 13 others. When the explosion occurred the bottom of the converter dropped out, throwing 15 tons of molten metal into the pit where the 15 men were working with their ladles.

AN EXCURSION TRAIN on the Southern Ry. running between Cleveland, Ohio, and the Florida resorts, broke through a trestle over Copper Mine Creek, about 50 miles north of Atlanta, on Jan. 7, and was completely wrecked. The engineer and fireman and another trainman were killed and about eighty passengers more or less seriously injured. The engine had already passed over the trestle, but just on the other side left the tracks and turned over. This pulled the baggage car over and toppled five Pullmans off the trestle into the creek.

A UNION PASSENGER STATION at Tacoma, Wash., is proposed by the Northern Pacific R. R. The main building would be 123 × 200 ft., and three stories high. The building would have a floor space of nearly 50,000 sq. ft. and would be built of steel and concrete, faced with brick and trimmed with stone. Its estimated cost is $1,200,000.

BOULEVARD AND PARKWAY PLANS for Denver, Colo., have been prepared by Mr. George E. Kessler, of Kansas City. When completed the system laid out would connect all the outlying parts of the city with its center by means of parkways and boulevards. Some of these, in conjunction with existing streets, would be on the diagonal plan.

A NEW YELLOWSTONE PARK APPROACH now being built by the Oregon Short Line will be ready for operation by next season. The line extends from St. Anthony, Idaho, over the continental divide at Rea's Pass (7,000 ft. elevation) 70 miles to Yellowstone, Mont., at the western edge of the park. A handsome station building, modeled after the famous Old Faithful Inn, has been designed for the Yellowstone terminal.

A LARGE FORE-AND-AFT SCHOONER YACHT with auxiliary engines is being built for Mr. W. Harry Brown of the New York Yacht Club, for a three-year exploration cruise. The yacht, which is claimed to be the largest of its class in the world, will measure 197 ft. 6 ins. over all, with a 150-ft. water line and a 32-ft. beam. She will be built of steel and will have a displacement of 800 tons and a coal capacity of 200 tons. The cost will probably be over $300,000.

FIRE PROTECTION must be maintained on all high buildings in course of construction in New York City, according to the following recent orders of the Fire Commissioner:

In all buildings in course of erection the standpipe (fire line) must be carried up with each floor, after the structure reaches the seventh floor, or a height of 85 ft., and an outside Siamese connection provided in proper and accessible place, and regulation hose outlets provided on each floor above the second as the work progresses, the top level of riser to be securely capped at all times except when work on the standpipe is in progress at that point.

THE BATTERY TUNNEL UNDER THE EAST RIVER at New York City will be in regular service by the time this issue is distributed. Operation is scheduled to begin early on Thursday, Jan. 9. The tunnel is part of the Rapid Transit Subway of New York City, and connects the Manhattan Borough system with the lines in Brooklyn Borough. The latter are only in small part completed, and trains will be run only to Borough Hall for the present. It is planned to run the trains of the Bronx branch of the subway through the tunnel to Brooklyn, whereas the trains of the west side or Kingsbridge branch will turn at the Battery loop as heretofore.

The Hudson River tunnel of the Hudson & Manhattan R. R. Co., at Morton St., New York, which is to be opened inside of two months, is now so far advanced that on Saturday, Jan. 4 the first electric train was run through from Hoboken to New York. Work on the city terminal section on Sixth Ave., and the terminal station at 33d St. and Sixth Ave., is progressing rapidly.

THE HUDSON TUNNELS of the Hudson and Manhattan Railway Co., at New York City, will be opened for service between Hoboken and the station at Fourteenth St. and Sixth Ave., New York, by March 7, 1908. The company is prepared to operate its Morton St. tunnels under the Hudson River (the "Old Hudson River Tunnel") by January, but this would bring New Jersey passengers only to Christopher and Greenwich Sts., practically the New York shore end of the tunnel, which is not a traffic center and would serve but few passengers. As the permit of the company required it to begin operating the river tunnel by Jan. 7, 1908, Mr. W. G. McAdoo, president of the company, has applied for an extension of time to March 7, promising that by that date the line could be opened (via. Christopher St. and Sixth Ave.) to Sixth Ave. and Fourteenth St., a busy retail store center. The permit was granted by the Public Service Commission a week ago. The other Hudson River tunnels of the company, at Fulton and Cortlandt Sts., New York, two miles south of the Morton St. tunnels, are rapidly nearing the stage of meeting of the shields. Their New York terminal, a pair of immense office and station buildings on Church St., between Cortlandt and Fulton Sts., is already completed as to the train floor, and the 22-story superstructures are so far advanced that they will be ready for occupancy by next spring. These lower tunnels will connect with the upper or Morton St. tunnels by a pair of north-south tunnels through Jersey City and Hoboken, and will also have a surface extension to the tracks of the Pennsylvania Railroad between Jersey City and Newark.

PATENTED PAVEMENTS AND PAVEMENT MATERIALS may be used in the City of New York when placed in competition with other pavements or pavement materials, according to a decision of the New York State Court of Appeals, rendered late in December, 1907. The decision is virtually an interpretation of Sec. 1554 of the charter of New York City, which reads:

Except for repairs, no patented pavement shall be laid and no patented article shall be advertised for, contracted for or purchased, except under such circumstances that there can be a fair and reasonable opportunity for competition, and conditions to secure which shall be prescribed by the Board of Estimate and Apportionment.

Under this section certain sheet asphalt paving interests have tried to prevent the use of bitulithic pavement, or an improved tar macadam, in New York City. The decision terminates five years of litigation and besides opening the way for bitulithic permits the use of any other patented pavement, provided only bids for it are submitted in competition with those for other pavements.

THE PANAMA CANAL LOCKS are to remain at the present projected width of 100 ft., according to press dispatches from Washington of Jan. 5, provided the unanimous opinion of the Canal Commission controls. The Navy Department may overturn this decision, but the present width is deemed by the Commission ample for all commercial requirements.

PANAMA CANAL WORK DURING DECEMBER once more broke all previous records. The total excavation was 2,200,539 cu. yds. place measurement, against 1,836,486 cu. yds. in November and 1,868,729 cu. yd. in October. The average daily December output for 25 working days was 88,000 cu. yds. against 76,600 cu. yds. in November for 24 days. The rainfall was only 3.85 ins. against 10.73 in November. The excavation records for November and December are summarized as follows:

	December. cu. yds.	November. cu. yds.
Steam Shovels:		
Culebra	1,000,026	778,860
Gatun locks and spillway	160,795	162,622
Mindi, Chagres and La Boca	152,117	92,237
Dredges:		
Colon	428,053	417,297
La Boca	349,551	365,423
Total, Canal Prism	2,090,542	1,816,439
Accessory works outside the Canal Prism	103,997	22,047
Total excavation	2,200,539	1,838,486

THE SOUTHWEST PASS OF THE MISSISSIPPI River will be available as a new deep water channel of the river within the next few weeks, upon the completion of the new jetty work which has been in course of construction for the past four years. These jetties are larger than the old ones at the South Pass which have maintained the channel that has been for about thirty years the commercial entrance to the river. The new jetties are parallel walls lying about half a mile apart and about four miles long. They have produced a channel of a minimum depth of 35 ft. and width of 1,000 ft. All that now remains to be done is some minor dredging in and about the channel. The problem of the Southwest Pass channel was quite fully treated in Engineering News for Oct. 4, 1900, p. 222, and Oct. 11, 1900, p. 250.

THE CONNECTICUT AVENUE CONCRETE BRIDGE at Washington, D. C., was opened for traffic on Jan. 4. This structure (Eng. News, June 1, 1905, p. 571), probably the largest concrete bridge in the world, carries Connecticut Ave. across the deep wooded gorge through which flows Rock Creek and which separates the present residence section of the northwestern part of the city from the suburban subdivisions. It consists of five 150-ft. and two 82-ft. full centered circular arches, constructed throughout of non-reinforced mass concrete relieved at corners, cornices and in the arch ring by concrete block molded on the ground with a crushed granite mortar face and tool dressed to shape and line. The roadway, which at the deepest part of the gorge is 135 ft. above the creek level, is 1,341 ft. long and 50 ft. wide,

with a carriageway of 35 ft. and two sidewalks of 7½ ft. each. No provision has been made for immediate use by street railways although the stresses were computed under the assumption of such a load. The bridge is the design of the late George S. Morrison and has been erected by the Engineer Department of the District of Columbia under the direct supervision of Mr. W. J. Douglas, M. Am. Soc. C. E., Engineer of Bridges.

THE UTILIZATION OF SAWMILL WASTE in paper making is touched upon in a report from Consul L. E. Dudley, of Vancouver, B. C., published in the U. S. Consular Reports. He states that a company has been organized in that city and capitalized at $1,000,000. This company has arranged for the purchase of 80 acres opposite Gambier Island on Howe Sound, 20 mi. from Vancouver, at the entrance of Rainy River, where there is an abundant supply of water at all seasons. By utilizing the water power of the Rainy River, the company will develop sufficient water power for mechanical and domestic use, and for a series of mechanical grinders for the manufacture of ground wood, which is used to supplement chemical fiber in the manufacture of paper. The plan is to convey the refuse of the local mills in this vicinity, almost entirely in the form of sawdust, in scows to the plant, where the entire mass is to be disintegrated into suitable fineness for conversion into wood pulp. One of the innovations of the new company is a patented process by which it is claimed that Douglas fir and other resinous woods can be used in the manufacture of wood pulp. By the new process, it is claimed, all the resinous products are passed off in vapor and the pure fiber is recovered by pressing the pulp residue. The company has in operation a complete model plant which demonstrates this process. The projected plant on the Rainy River, near Vancouver, will have a capacity of 360 tons of finished material per week, consisting of 200 tons of news and 160 tons of manila, wrapping, box boards, building paper, etc.

THIRTY-FIVE PREPAYING-TYPE STREET CARS were placed in service on the Niagara St. line of Buffalo, N. Y. traction system, on Jan. 6. A report in the "New York Times" states that:

> The public seemed to take kindly to the cars, the requisite nickel being ready in every instance. At Shelton Square the company has erected a kiosk at which change is made and tickets are sold, thus facilitating things generally. It is estimated that fully 10,000 persons were transported in the new cars during the rush hours between 5.30 and 6.15, and in not one instance was there even a momentary hitch. During the rush hours tonight, when Shelton Square was packed with home-going people, the cars were filled one-third quicker than the cars of the old pattern running on the same tracks. The time schedule of the line has been reduced 7 minutes on the original run of 41 minutes. The cars are patterned after those in use in Montreal, and can hold 80 passengers, 50 of them seated.

PERSONALS.

Mr. C. A. Goodnow has resigned his position as General Manager of the Chicago & Alton Ry.

Mr. George W. Tonson has been elected President of the Board of Public Service of Toledo, Ohio, to succeed Mr. Oscar Sabin.

Mr. F. A. Rapp, recently constructing engineer with the Stuart-Beebe Co., of Pittsburg, has been appointed Instructor in Civil Engineering at Idaho State University.

Mr. D. G. Seymour, Resident Engineer in charge of the Stevenson, Ala., extension of the Southern Ry., has resigned to open an office in Chattanooga, Tenn., as Consulting Engineer.

Mr. J. J. Cozzins has been promoted to Chief Superintendent of Signals for the N. Y. C. & H. R. R. R. on the Buffalo, Rochester, Western Rome, Watertown & Ogdensburgh and Pennsylvania divisions.

Mr. William T. Blunt, M. Am. Soc. C. E., lately Superintendent for G. H. Brayman & Co. on the Boston Harbor work, has been appointed Superintendent of Subaqueous Rock Excavation on the Panama Canal.

Mr. H. M. Church, formerly Assistant Engineer of Maintenance of Way at Cincinnati, Ohio, on the Baltimore & Ohio Southwestern Ry. Co., has been appointed Division Engineer at the same place, vice Mr. L. F. Boeh, resigned.

Mr. Weston E. Fuller, Assoc. M. Am. Soc. C. E., has been admitted to the firm of Hazen & Whipple, Consulting Engineers, 220 Broadway, N. Y. The firm now consists of Mr. Allen Hazen, M. Am. Soc. C. E.; Mr. George C. Whipple, M. Am. Soc. C. E.; Mr. L. N. Babbitt and Mr. W. E. Fuller.

The engineering firm of Sellers & Rippey, of which the late Dr. Coleman Sellers was senior member, will be continued under the same firm name by the surviving members, Mr. S. Howard Rippey, M. Am. Soc. C. E., and Mr. H. W. Sellers. Their office is in the Stephen Girard Building, Philadelphia.

Mr. Edgar Van Etten, who, up to a few months ago, was the Vice-President of the New York Central & Hudson River R. R., in active charge of the Boston & Albany R. R., has resigned his position as Vice-President to become President of the Cuba Eastern R. R. and allied properties. Since the reorganization of the Boston & Albany R. R. in October last, Mr. Van Etten has had only a nominal connection with the N. Y. C. & H. R. R. R.

Capt. John E. Pillsbury, U. S. N., has been appointed Chief of the Bureau of Navigation to succeed Rear-Admiral W. H. Brownson, whose resignation was announced last week. The appointment of Commander C. McR. Winslow to the position, as recorded in our last week's note, was merely temporary. Capt. Pillsbury's last command was the dynamite cruiser "Vesuvius" during the Spanish war. Since that time he has had service at the Boston navy yard and on the General Board of the Navy.

Mr. Charles V. Weston, M. Am. Soc. C. E., has been elected President and General Manager of the South Side Elevated Ry., of Chicago. He was Chief Engineer of the road from 1903 until May, 1907, when he resigned to take the position as the city's representative on the Board of Supervising Engineers in charge of the reconstruction of the street railway system. The three-tracking and general reconstruction and improvement of the elevated railway was done under Mr. Weston as Chief Engineer.

Obituary.

Commander Henry H. Hosley, U. S. N., Supervisor of the Harbor of New York, died in that city on Jan. 6, at the age of 55 years. Commander Hosley came into public notice when he took the dry dock "Dewey" and its convoy fleet on the famous voyage from the Virginia capes to Manila, via the Suez Canal, a distance of 11,000 miles. The trip made a record of seamanship that called forth a great amount of praise for the commanding officer, both from the officials of the Navy Department and naval experts the world over. Commander Hosley never quite recovered from the strenuous exertion of the six months' transoceanic voyage.

Prof. C. A. Young, a past Professor of Astronomy at Princeton University, died in Hanover, N. H., on Jan. 4, at the age of 72 years. Prof. Young was one of the most noted astronomers of this country and had served on the faculties of Western Reserve College, 1857-66; Dartmouth, from which he graduated in 1853, 1866-77, and Princeton, 1877-1905. He was a specialist in solar physics and most of his research work was along that line. His text-books on The Sun, General Astronomy, The Elements of Astronomy, etc., are very well known. Besides his first degree from Dartmouth, Prof. Young held honorary degrees from the University of Pennsylvania, Hamilton College, Wesleyan, Columbia, Dartmouth and Western Reserve.

Mr. George W. Rafter, M. Am. Soc. C. E., a consulting engineer on water supply and power and on sewage disposal, died on Dec. 29, 1907, at Carlsbad, Austria, where he had but recently gone from Rochester, N. Y., to regain his health. Mr. Rafter was born in Phelps, N. Y., Dec. 9, 1851, and was educated in the primary schools of the state and Cornell University. His first professional work of note was in connection with the construction of the Rochester water-works in 1876. From 1876 to 1880 he was connected with railways in Texas and the southwest, but in 1880 he resumed his water-works practice as constructing engineer of the systems at Fort Worth, Texas, and later at Fredonia, Rochester, Westfield and elsewhere in New York State. In 1894 he became connected with the State Engineer's office of the State of New York and in that capacity made a number of reports on lift bridges in Europe and on the water resources of New York State. He also made some extensive researches in the flow of water over weirs, and in the strength of concrete. Earlier Mr. Rafter devoted considerable time to the biology of water and devised methods for use in the microscopical examination of water. He had one of the most extensive engineering libraries in the United States. Besides contributing more than a hundred papers and discussions to technical societies and writing several small books on water, sewage, etc., Mr. Rafter was senior author, with Mr. M. N. Baker, of the editorial staff of this journal, of "Sewage Disposal in the United States."

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 15. Annual meeting at New York City. Secy., Chas. Warren Hunt, 220 West 57th St., New York City.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 15-17. Annual meeting at Champaign, Ill. (University of Illinois). Secy., E. E. R. Tratman, 1636 Monadnock Block, Chicago.

INDIANA ENGINEERING SOCIETY.
Jan. 16-18. Annual meeting at Indianapolis. Secy., Charles C. Brown, 408 Commercial Club Building, Indianapolis.

NATIONAL ASSOCIATION OF CEMENT USERS.
Jan. 20-25. Fourth annual convention at Buffalo. President, Richard L. Humphrey, 1001 Harrison Bldg., Philadelphia, Pa. Convention Manager, Dan H. Lewis, 760 Main St., Buffalo.

WOOD PRESERVERS' ASSOCIATION.
Jan. 21. Fourth annual meeting at Kansas City. Secy., C. W. Berry, Laramie, Wyo.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 21-23. Annual meeting at New York, Engineering Societies Building, 29 West 39th St. Secy., W. M. Mackay, 29 West 39th St., New York.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Jan. 23-25. Annual meeting at Chicago. Secy., C. R. Richards, Columbia University, N. Y.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 28-30. Annual meeting at Montreal. Secy., Prof. C. H. McLeod, 877 Dorchester St., Montreal.

NATIONAL BRICK MANUFACTURERS' ASSOCIATION.
Feb. 3-8. Annual meeting at Columbus, Ohio. Secy., Theo. A. Randall, Indianapolis, Ind.

AMERICAN CERAMIC SOCIETY.
Feb. 3-5. Annual meeting at Columbus, Ohio. Secy., Edw. Orton, Jr., Columbus, Ohio.

CONNECTICUT SOCIETY OF CIVIL ENGINEERS.
Feb. 11-12. Annual meeting at New Haven. Secy., T. F. Jackson, New Haven.

OHIO ENGINEERING SOCIETY.
Feb. 11-13. Annual meeting at Columbus. Secy., E. G. Bradbury, 85 North High St., Columbus.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 18. Annual meeting at New York. Secy., R. W. Raymond, 29 West 39th St., New York.

THE BROOKLYN ENGINEERS' CLUB have elected the following officers for the coming year: George C. Whipple, Pres.; Joseph Strachan, Secy.; Wm. T. Donnelly, Treas.

THE ENGINEERS' CLUB OF PHILADELPHIA have recently purchased a new club-house, located at No. 1317 Spruce St. It is a four-story building which, when altered, will be amply sufficient for the needs of the society both in a social and business way.

THE ENGINEERS' CLUB OF CINCINNATI elected the following officers at its 20th annual meeting in December, 1907. H. C. Innes, Pres.; E. A. Gast, Secy. and Treas. The club received 50 new members during the past year and now has a total membership of 130. It is very active and is meeting with a great amount of success in the territory which it covers.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.—The regular monthly meeting will be held in the auditorium of the Engineering Societies Building, 33 West 39th St., New York City, on Friday, Jan. 10, at 8 p. m. The following papers will be presented: (1) "The New Haven System of Single-Phase Distribution, with Special Reference to Sectionalization," by W. S. Murray, Electrical Engineer, New York, New Haven & Hartford Railroad. (2) "A Single-Phase Railway Motor," by E. F. Alexanderson, Electrical Engineer, General Electric Company, Schenectady, N. Y.

A special meeting will be held on Friday, Jan. 24, at 8 p. m., at which the following papers will be presented: (1) "Electrical Engineering Education," by Charles P. Steinmetz, Chief Electrician of the General Electric Company, Schenectady, N. Y. (2) "The Best Engineering Education," by Charles F. Scott, Consulting Engineer of the Westinghouse Electric & Mfg. Co.

A memorial service in honor of the late Lord Kelvin will be held under the auspices of the American Institute of Electrical Engineers, in the auditorium of the Engineers' Building, 33 West 39th St., New York, on Sunday, Jan. 12, 1908, at 3.30 p. m. All the national learned societies have been invited to participate in these exercises.

THE SUPERVISING IRON ERECTORS OF AMERICA.—With the development of structural iron and steel work within the past two decades there has grown up a branch of building engineers who are specialists in their line and who are engaged in work entirely different from the standardized forms of the recent past. Of these specialists there are three classes: the iron manufacturers, the designers and the erectors or constructors. The first two classes have been well represented in societies and associations for many years, but the last one, the erectors, have not been so well organized. However, some four years ago in New York City there was started a society of such engineers, known as the Supervising Iron Erectors of America, which represented all that was good and stable in that branch of the profession. This society is now in a flourishing condition. It holds semi-monthly meetings on the second and fourth Wednesday of every month at its rooms in the Grand Opera House, Eighth Ave. and 23d St., New York City, and after the transaction of regular business informal receptions and smokers are held. The members of the association are ambitious for its technical progress and recognition and extend a hearty invitation to all engineers and bridge and structural men to visit the rooms and attend the meetings. It is also the aim and object of the society to furnish the employers at all times with competent and well trained superintendents and foremen for the erection of bridge and structural steel work in this or foreign countries. The present officers are: A. M. Harvey, Pres.; P. V. Sholts, Treas.; E. H. Kampff, Chm. of Board of Directors, and M. V. B. Ruland, 761 Jefferson Ave., Brooklyn, Secy.